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The accompanying view of the Link-Belt Co.'s drafting room shows some of the 100 Universal Drafting Machines used by this concern.
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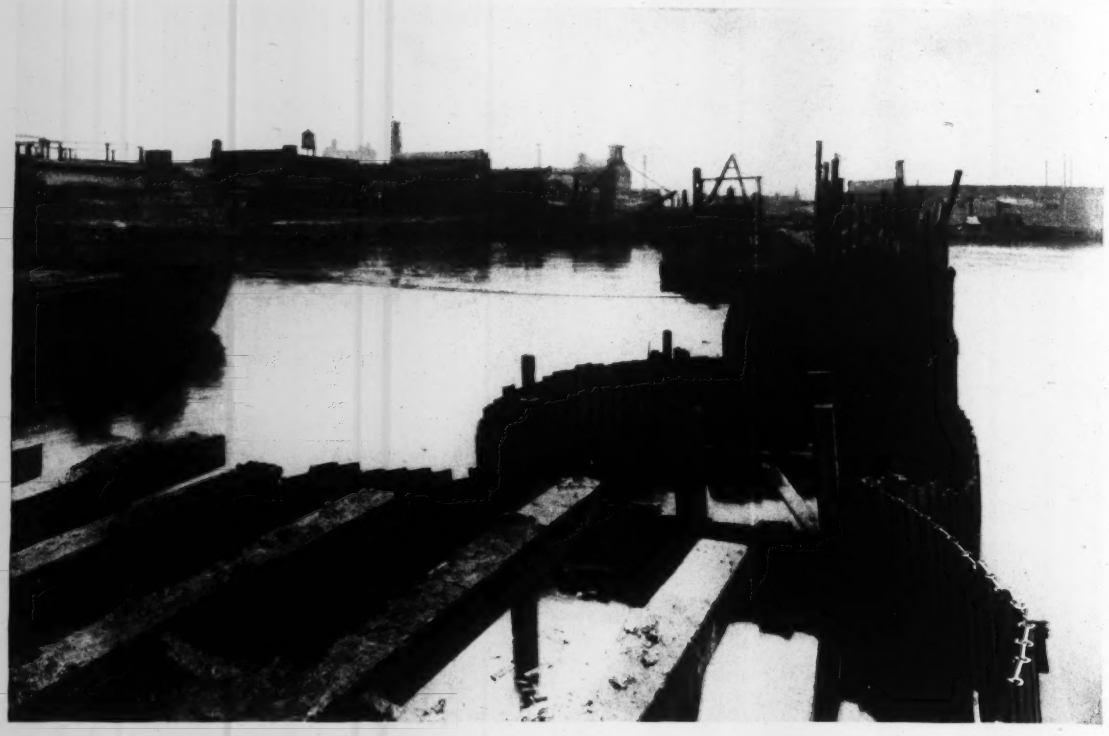

Pockets on the River Side of the Lackawanna Steel Sheet Piling Cofferdam, 46th Street Piers, N. Y.

All steel sheet piling here was of the $12\frac{3}{4}$ by $\frac{3}{8}$-in. Lackawanna 35-pound Straight-Web Section, in pieces of a maximum length of 73 feet, for a penetration of 34 feet and projection of $8\frac{1}{2}$ to 9 feet above mean low water level.

The steel sheet piling driven in that part of the cofferdam which was in pocket form, was assembled around wooden templates, each fastened to four wooden piles (see smaller picture.) Assembling was accomplished by the 100-foot spliced and trussed wooden boom of a floating derrick.

The sheet piling sections usually settled under their own weight alone to within about ten feet of the rock surface, and were then driven in sets of two or three to moderate refusal, after which they were driven singly to final refusal by Monarch No. 2, Hamilton and Arnott steam hammers.

Soft material tubbed out from inside the cofferdam was placed within the pockets to displace water and to render the sheet piling joints watertight. Heavy filling was also deposited within the pockets to help in resisting thrust, and this additional filling was brought in from shore by wagons over a plank roadway extending over the pockets as fast as the filling progressed. Stringers for extension of this roadway are shown in the left hand corner of the upper picture.

The progressive steps in making this cofferdam stable will be explained in next week's advertisement.

And steps in other successful cofferdam work are explained in our book "Lackawanna Steel Sheet Piling," which will be sent to any contracting or managing engineer.

LACKAWANNA STEEL COMPANY

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Engineering News

VOLUME 75 MARCH 2, 1916 NUMBER 9

Galveston Storm Repairs

By N. T. Blackburn*

SYNOPSIS—Damages to engineering structures caused by the Galveston storm and flood of Aug. 16 and 17, 1915, are outlined. Progress on repairs to the sea wall and its sand backing, to the boulevard, and the water-supply conduits from the mainland are reviewed.

The West Indies hurricane that passed over Galveston, Tex., Aug. 16 and 17, 1915, was in some respects of greater violence and intensity than any other in the history of the place. While the greatest wind velocity was less than in 1900, the period of high winds was greater, and the water backed up on the island from Galveston Bay remained longer. It was due to this fact that oak trees 40 and 50 years old, which had survived many storms and withstood the grade-raising operations, were killed. There is no doubt in the mind of anyone who went through the storm that the sea wall saved the city. The only damage done to the concrete of the sea wall was the knocking off of two small chips of about 2 cu.ft.

*United States Assistant Engineer, Galveston, Tex.

capacity each from the top of the wall at the foot of 39th St., when the four-masted schooner "Dora Allison" was dashed on it during the height of the storm. This schooner, which was bound from Progreso, Mex., to Mobile, Ala., loaded with sisal, encountered the hurricane in the Gulf and was blown off her course. When she struck the sea wall, she was dragging two anchors. These caught in front of the wall, or under its toe, and the schooner fetched up and pounded to pieces on top of the wall, the fragments of hull, masts, cargo, etc., being scattered over and behind the Fort Crockett military reservation. This occurrence is considered a rather remarkable test of the sea wall.

The riprap apron in front of the wall was undermined in many places and dropped, allowing the sand beach to be scoured away (for a depth of 3 ft. in the extreme case) under the toe of the wall, thus exposing the sheet-pile foundation to the action of the teredo.

The first efforts made after the storm, both on the part of the county and the United States Engineer Office, were directed toward temporary repairs, only, to their respective portions of the sea wall. Such small riprap

FIG. 1. TEMPORARY RIPRAP, SAND AND SHELL PROTECTION FOR PORTION OF GALVESTON SEA WALL

as could be gathered up, broken concrete, sand and shell were placed in the front of the wall, to fill in the holes and protect the sheet piles from the teredo (Fig. 1). These temporary repairs have now been completed, and the county is carefully placing about 24,000 tons of additional riprap as an apron, to give permanent protection along the toe of the wall.

The boulevard and sidewalk behind the sea wall were more or less damaged along their entire length. The worst damage was from 39th to 43rd St. in the Fort Crockett military reservation, and from about 6th to 19th St. in the city, where the boulevard and sidewalk were nearly or quite washed away, and with them the sand-fill behind the wall, to a maximum depth of about 10 ft. This sand was washed several blocks back from the wall, filling streets and yards. In this latter area many houses were undermined and tilted, or turned over.

The repairs in the city up to the middle of January, 1916, have been confined to straightening up houses, gathering brick from the damaged boulevard for relaying, repairing the smaller breaks in the boulevard and preparing to begin construction of a concrete sheet-pile bulkhead behind the sea wall under the new plan. The new plan consists briefly in placing a boulevard, or roadway, 100 ft. wide back of the sea wall, beginning at 6th St. where the wall turns back across the island. Sixteen feet of this roadway will be a sidewalk, and the back line of the boulevard will be El. +19, or 2 ft. above the top of the sea wall. The boulevard is to be flush with the top of the wall, with an interrupted curb 4 in. high between them, to allow free drainage of water from the boulevard back over the sea wall. Along the back side of the boulevard a continuous line of reinforced-concrete sheet piles 6 in. thick and 14 ft. long is to be placed, capped with concrete coping and tied back at 10-ft. intervals with iron rods to an 8x12-in. continuous concrete deadman, buried under the pavement 20 ft. from the sheet piles.

Along the wharf front the damage consisted principally in the scattering of ships, dredges, barges and boats of all kinds around Galveston Bay. One ship grounded in the railroad yards just west of the city; two on the mainland south of Texas City; two on the dredge spoil-bank on the north side of Galveston Channel; two on Pelican Island, in Galveston Bay; and one on Red Fish Reef, some 12 mi. up Galveston Bay from her anchorage. These ships have all been dredged off and repaired. Most of the smaller boats, barges, dredges, etc., have also been floated and repaired. Some will never be recovered.

The damage to the wharf system itself was confined principally to the entire or partial wrecking of wooden warehouse sheds on the wharves. One steel elevator conveyor was blown down and wrecked, the banana conveyors of the United Fruit Co. were twisted and damaged, and the tracks, wharves, etc., were littered with wreckage of every description. Repairs to all this damage have been nearly completed and the wreckage cleaned up.

Restoring Railroad Connection and Water-Supply

Probably the most serious blow to the city and the railroads was the wrecking of the causeway across West Galveston Bay, cutting off communication with the mainland and incidentally destroying the fresh-water main that was laid from the source of supply on the mainland. The earth portions of the causeway, which lay on each side of the central arch- and lift-bridge portion and which consisted of a sand mound, or fill, between parallel rows of concrete sheet piles at the toes, with concrete slabs on the sloping sides, were washed down. The railroad and interurban tracks and the duplicate water main were carried away also, and the county highway was destroyed.

FIG. 2. DRIVING LAST BENT FOR TEMPORARY RAILROAD TRESTLE OVER WRECKED PORTION OF GALVESTON CAUSEWAY

Within two weeks after the storm the Santa Fe and Southern Pacific railroads, working in conjunction, had built a trestle (Fig. 2) across the gaps and reëstablished railroad communication between the city and the mainland, the first train passing over the trestle Sept. 1.

As a fresh-water supply was imperatively needed, an 8-in. temporary main was laid for the city by the railroad forces engaged on the building of the trestle. This main is shown at the left-hand side of Fig. 2. It covered only the wrecked part of the main, that part over the concrete arches being undamaged. Efforts were then directed by the city toward repairing the broken main. This work has finally been completed, and the city is now in possession of its regular fresh-water supply.

As the old water main, which was buried in the bed of the bay alongside the causeway and was in use before the building of that structure, was undermined and also wrecked during the storm, plans have been made for a

30-in. emergency main to be laid from 4 to 8 ft. below the present bed of the bay. The bottom of this main will be 17.8 ft. below mean low tide in the middle of the bay opposite the lift span of the causeway, sloping up to about 5 ft. below the same datum at each shore line, where it will connect with the old main.

A bond issue providing funds for this project was carried in a special election on Jan. 18. It is understood that with the rebuilding of the causeway the duplicate main in this structure will be replaced. It is believed that with these mains and the emergency main in existence all danger of an interruption of the water-supply due to a storm will be averted.

Plans for Rebuilding Causeway to Mainland

Plans for the rebuilding of the causeway are not yet decided on. The structure was owned jointly by the County of Galveston, the steam railroads and the interurban electric road entering the city. Engineers representing these various interests have met and are meeting together frequently to adjust their differences of plan and agree on a plan of rebuilding acceptable to all. While it at first appeared that there was a deadlock, the last few meetings have indicated that differences are rapidly being adjusted, as all parties are making concessions and it now appears that a plan will soon be agreed upon. When finally agreed on the plan will be submitted to the general managers of the railroads and the interurban and the county representative for ratification.

The county favors the plan of extending the concrete arches entirely across the bay, and this plan is strongly urged by the Citizens Committee of Galveston. As the preliminary estimate for the extension of the arches was about $2,000,000 a plan was at first considered of replacing the wrecked part by an open creosoted-pile trestle, the county highway being on an independent trestle with ballasted deck.

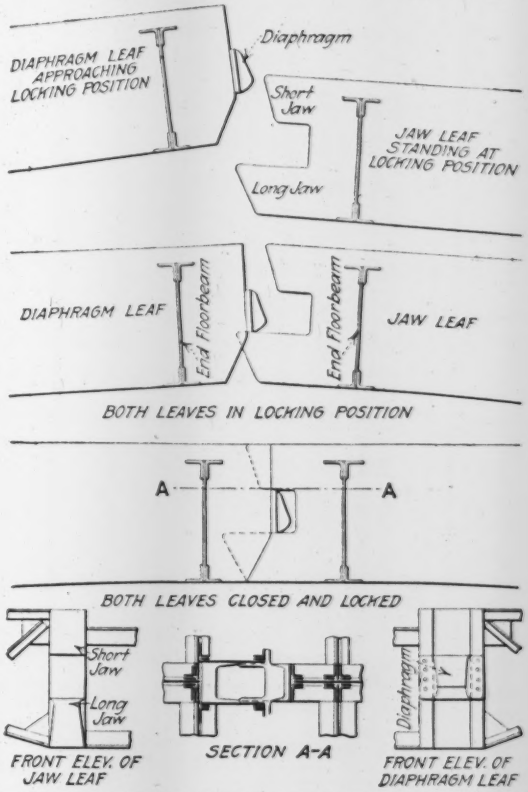
Another plan was to build a reinforced-concrete pile trestle. It is understood that on reëstimate and more thorough consideration it was believed by the designing engineer of the old causeway, William Mueser, who has been acting as consulting engineer for the county, that the arches could be extended to shore on each side for $1,500,000 by using shorter spans than the present arch-bridge part and filling in between the concrete sheet-pile bulkheads along the toe of the earth portion (which are the only parts standing of this portion) so as to make dry land on which to work in constructing the arches.

As these pile bulkheads project above the surface of the water about 4 ft. and extend from the shore out they form obstructions and restrict the free movement of the water. One of the remaining points to be settled is whether any part of these bulkheads shall be torn out so as to give more free passageway for the water, the railroad engineers favoring this for at least a part of the bulkheads. There seems to be general agreement that an open structure allowing free movement of the bay water is essential during storms and that no part of the earth portion shall be rebuilt in its old form. The question of plan is largely a financial one, but the indications are for the plan of extending the arches entirely across the bay.

The plans under discussion provide for a causeway 66 ft. in width, conforming to the arches now standing. Of this width the interurban company asks for 15 ft., the county 22 ft. and the four steam railroads 29 ft.

Bascule-Bridge End Lock

In the recent bascule bridges built by the Scherzer Rolling-Lift Bridge Co., of Chicago, the use of moving end locks and their mechanism is dispensed with. As the leaves come together in closing, the ends of the girders of the opposite leaves are connected and locked automatically by means of what is termed a shear lock. This consists of a diaphragm on the end of one girder which engages with a recess formed between an upper (short) and lower (long) jaw on the end of the opposite girder.



AUTOMATIC END LOCK FOR SCHERZER ROLLING-LIFT BASCULE BRIDGES

In closing the bridge the leaf with jaw is lowered to such a position that the shorter (upper) jaw will be cleared by the diaphragm on the other leaf. The leaf with diaphragm is then lowered until the diaphragm engages the longer (lower) jaw, after which both leaves are lowered to their final position. This last motion, due to the longitudinal movement or advance of the leaves in their rolling travel, brings the parts into their full locked position, with the diaphragm between the upper and lower jaws.

In opening the bridge both leaves are raised simultaneously, disengaging the lock by the longitudinal movement due to the leaves rolling backward as they swing up. Dials in the operating cabin show the positions of the leaves at all times, so that it is an easy matter for the operator to control the movements. The diaphragms and jaws are steel castings.

With this construction no tail locks are required. The shear locks and track-plate bearings prevent the leaves from being lowered below their normal position. Bumping blocks which engage the counterweighted tail ends of the leaves check the travel at the full open position.

Building an Unusual Foundation for a Heavy Machine

By G. L. Knight*

SYNOPSIS—A spare mass foundation, useless for plant extension, owing to rapid advance in turbine design, had to be cut out. No room for new mass foundation, so a steel platform was erected on concrete-and-pile footings. Condensing-water tunnels interfered and one had to be bridged, so that heavy load would not come on it. Hollow piles driven by drop hammer; 720 hr. to drive 23 piles. Piles tested to 40 tons.

In providing for the installation of a 22,000-kw. turbo-generator at the Gold St. generating station of the Edison Electric Illuminating Co., of Brooklyn, N. Y., some very unusual conditions and difficulties were encountered which called for exceptional methods in overcoming them. The turbine is the last of six units in the station and is No. 3 in a row from the East River end of the building.

In 1911, when the adjacent 12,000-kw. vertical unit No. 2 was installed, it was intended to make No. 3 a duplicate of No. 2 and install it during the following year or not later than 1913. In view of this plan, the foundations for No. 3, below the basement-floor level, together with the connections to the condensing tunnels, were put in at the same time as those of No. 2. Conditions of load, however, made is unnecessary to purchase additional capacity until 1914. In the meantime steam-turbine design had undergone rapid changes, due principally to the increase in rotative speeds, which resulted in greater economy in steam consumption per kilowatt-hour and decreased cost per kilowatt of the entire unit.

It thus became possible to purchase a 22,000-kw. turbo-generator, at 1,500 r.p.m., the steam consumption of which was 12½ lb. per kw.-hr., in place of the 12,000-kw. vertical unit previously planned, which like No. 2 would have had a steam consumption of 15½ lb. per kw.-hr. at 750 r.p.m. In short the capacity of the unit was nearly doubled, the efficiency increased 20% and the cost per kilowatt decreased by about 30%. At the same time the vertical machine would fit into just the space required for the smaller unit. The principal dimensions of the new turbo-generator are as follows: Length overall, 45 ft.; width overall, 19 ft. 2 in.; weight of turbine, 241,000 lb.; weight of generator, 342,000 lb.; total weight, 583,000 lb.

The turbine required a condenser having 35,000 sq.ft. of surface; the condenser shell and tubes weighed, empty, 300,000 lb., and when completely filled with water, 618,000 lb.

The conditions in the station, with regard to condensing tunnels and adjacent apparatus, with the necessary requirements of accessibility and handling, made it necessary to install the condenser immediately under the steam turbine, with its long dimension parallel to the center line of the turbine shaft.

*Designing Engineer, Edison Electric Illuminating Co., 360 Pearl St., Brooklyn, N. Y.

As in the reaction type of turbine the clearances of the blading are very close, it is extremely necessary in planning a foundation for such a unit to make deflection negligible and to reduce the vibration to a minimum. For this purpose mass concrete for the foundation would have been best, but on account of the position of the condenser, the height available from basement floor to the main floor of the turbine room and the clearances necessary, it was found absolutely impossible to provide space for a concrete foundation. Structural steel was naturally the next available material and it was decided to erect a steel platform with very liberal allowances for factor of safety.

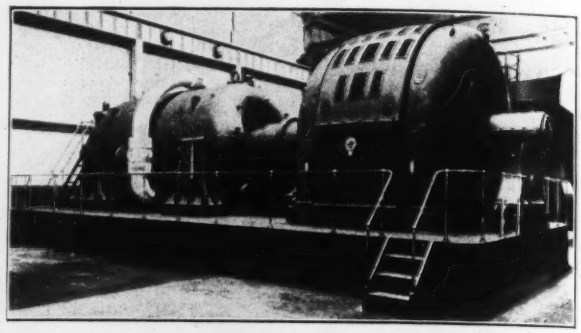


FIG. 1. 22,000-KW. TURBINE ON STEEL PLATFORM CARRIED ON AN INGENIOUS GRILLAGE SYSTEM
Area of unit, 45x19 ft.; weight of turbine, 120 tons; weight of generator, 171 tons

Condensing Tunnels Pass Through Site

There are two concrete condensing-water tunnels in the station, both of which pass through the area occupied by No. 3 foundation, as shown in Fig. 2. The easterly one consists of a condensing inlet and a discharge tunnel; the other tunnel is for inlet water only and extends some 24 ft. below the surface of the basement floor, hereafter referred to as datum. In the first case the discharge tunnel is over the inlet tunnel, each of them of horseshoe shape 8 ft. wide by 8 ft. deep. This structure was not strong enough to carry the load of the turbine foundation and machine; it therefore was necessary to build the foundation independent of the tunnel structure.

The westerly tunnel, built in 1912, is of reinforced-concrete construction lined with cast-iron plates. It is on good bottom, so that there was no danger in placing some additional weight on the concrete wall.

Referring to the cross-section, shown in Fig. 2, it will be seen that the structural design adopted consists of a pair of cantilever girders resting on a fulcrum of three girders supported by sectional tubular-steel piles filled with concrete. The cantilever girders carry in turn the middle columns of the turbine foundation at one end just beyond the fulcrum, and at the other end the two columns under the steam end of the turbine. The same girders support the condenser, as shown in Fig. 2. The columns supporting the outboard end of the generator are entirely independent of the rest of the foundation, because of its

being impossible to carry the foundation over the discharge-tunnel structure. In this case it was unnecessary to put in additional substructure, as there were good wooden piles in place; furthermore, the soil was sufficient to carry the load of 1½ tons per sq.ft., even had the piles not existed.

Cutting Out Existing Concrete Foundation

In constructing the foundation the first work to be done was to clear away and cut out all the mass concrete previously installed for No. 3 foundation. The mix was 1:3:5 (gravel).

The method consisted of drilling rows of holes 6 in. apart to the depth desired, using air and steam drills, and then breaking off in shelves with plug and feathers. The concrete, while hard and homogeneous, was very difficult to excavate. Instead of coming away in blocks, it shaled off in 3- or 4-in. layers, and in no case were the pieces as large as one-man stone. This fact rendered the progress of the work very slow, and the cost of excavation ran up to an almost prohibitive figure.

When excavation had been carried on sufficiently to permit piledriving, a machine was brought into the building and set up over the work, and driving began.

How the Piles Were Tested

When about half of the piles had been driven to the refusal stated, or work had been stopped on them on account of obstruction, it was decided to concrete two piles typical of existing conditions and test them with actual load, so as to be able to judge the necessity of going deeper with the remaining piles.

The piles were designed to carry a load of 20 tons each, and it therefore was desired to load them with at least 30 tons and if no settlement occurred, to continue the loading up to 40 tons. The test rig used is shown in Fig. 3. It will be seen from the drawing that the platform of I-beams on which the load rests is supported on jacks during the loading process. Provision is made to back off the jacks slightly at first for balancing, and afterward back away with 1-in. or more clearance of the blocking to allow for possible settlement. This precaution prevented accident from any shifting of the load.

Readings were taken with a level on an iron rod resting directly on the pilecap and protected from the load pigs by an iron pipe through which it passed freely. To the level rod was secured a piece of white paper on which a fine pencil line was drawn at the start of each loading, so that very accurate readings were possible.

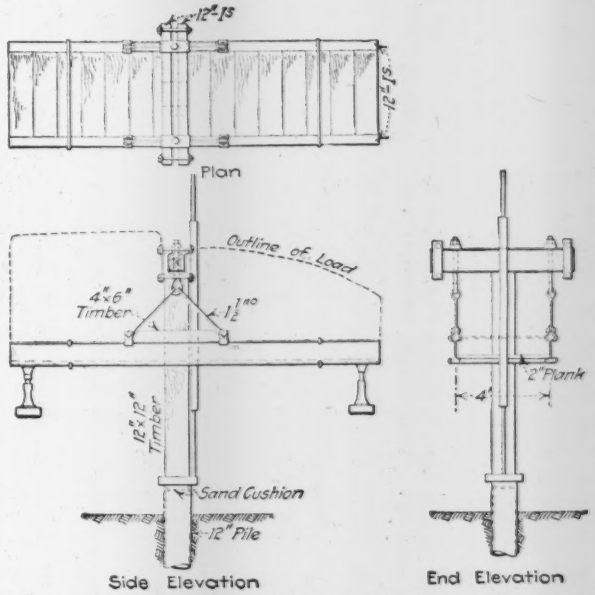


FIG. 3. GENERAL ARRANGEMENT OF RIG FOR TESTING THE PILES

The tables which follow show the results obtained on Piles 11 and 12. No. 11 pile had only one section, 18 ft. long, the driving having been stopped on account of the bottom of the pile turning over against a boulder or other obstruction and preventing the bucket from going through. As this pile was at the outer edge of the footing and extreme difficulty had been experienced in driving it, it represented about the worst condition of any of the piles, and a 25-ton test was considered sufficient.

No. 12 pile, on which the other test was made, was a very satisfactory one. The pile had been driven successfully to a depth of 21 ft. 7 in. below the excavation, or 34 ft. 7 in. below datum, in two sections of 18 and 5 ft. respectively, and the excavated material showed that it was fully 3 ft. into good sand bottom.

After the rig had been shifted, the second test was started at 30 tons. As there was no settlement at all the load was continued to 35 tons and then as settlement was inappreciable, being only $\frac{3}{32}$ in. in 24 hr., the load was continued to 40 tons, resulting in the very small settlement of $\frac{5}{64}$ in. In the interest of experience and because the rig proved so very practical the load was run up to 42½ and 45 tons with the results shown in the table. The test on pile 12 was so satisfactory that it was felt that ample factor of safety was provided, even making allowance for the piles driven to less than the full depth.

The loading material consisted of pigs of cast iron, each weighing about 42 lb. As they were borrowed for the purpose the tests were made at very reasonable cost.

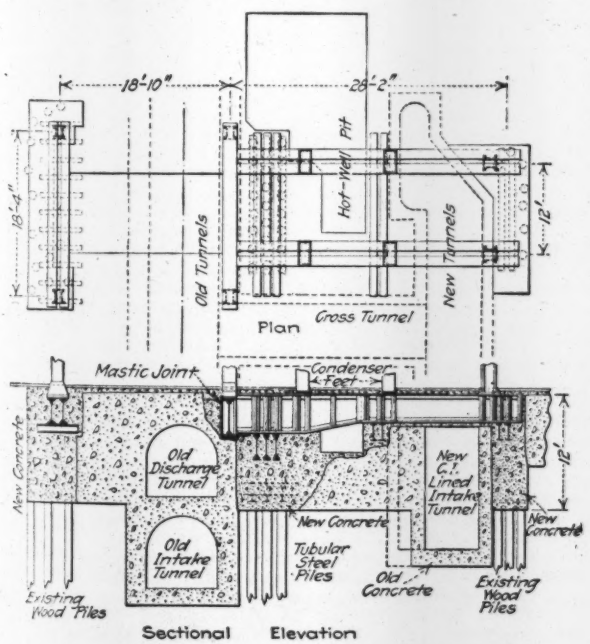
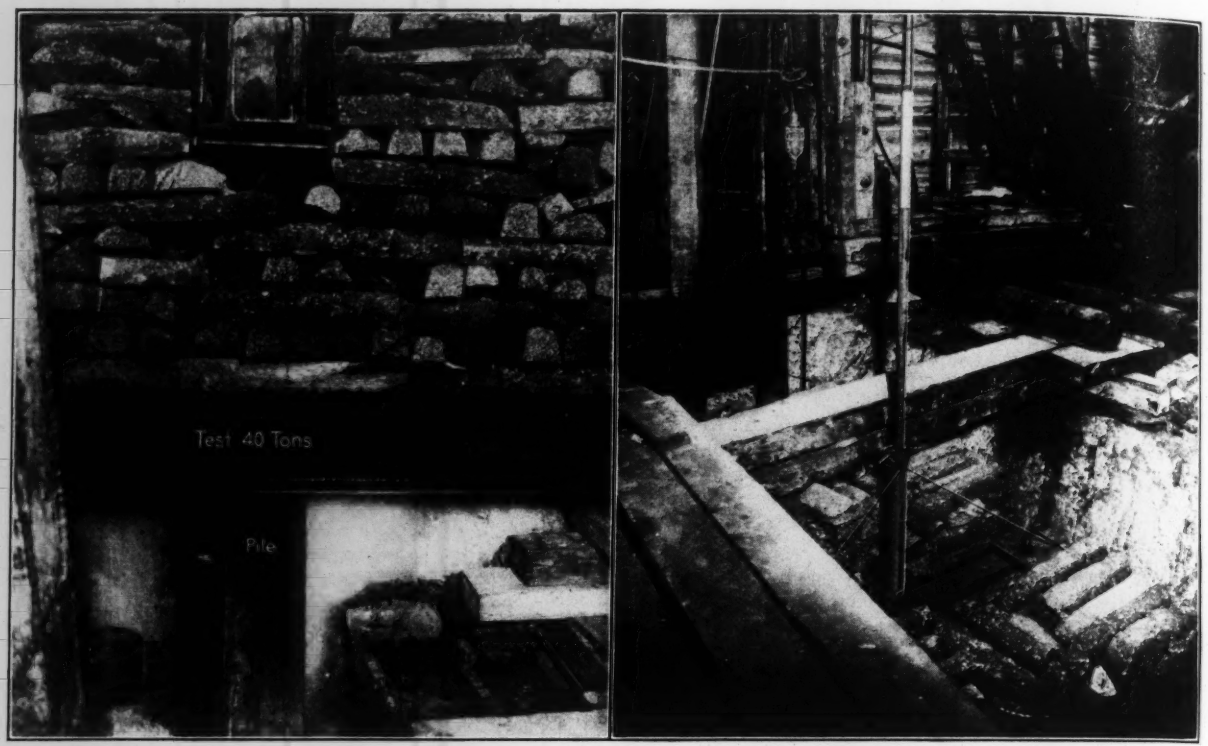


FIG. 2. FOUNDATION SUBSTRUCTURE, SHOWING CANTILEVER GIRDERS AT RIGHT



FIGS. 4 TO 5. (LEFT) METHOD OF SUSPENDING TEST LOAD. (RIGHT) LEVEL ROD WITH WHICH READINGS WERE TAKEN

From this point the placing of the piles was completed expeditiously. As soon as placed, they were dredged out with orange-peel buckets and filled with concrete. As the piles when driven contained water to depths varying with the condition of the soil at the bottom of the pile, the concrete had to be placed with a tremie, precaution

TABLE 1. LOADING TEST, PILE NO. 11

Date	Load, Tons	Time Load On	Time Load Off	Settlement
Oct. 25, 1914	25	11:00 a.m.		Down 11/64 in. on cap when first put on, due to compression.
Oct. 26, 1914	25		11:45 p.m.	Up 3/64 in.—not accounted for.

TABLE 2. LOADING TEST, PILE NO. 12

Date	Load, Tons	Time Load On	Time Load Off	Settlement	Remarks
Oct. 27, 1914	30.0	9:30 a.m.	3:10 p.m.	3/16 in. due to compression of sand cushion under cap when load was first put on.	No settlement thereafter.
Oct. 27, 1914	35.0	4:00 p.m.	7:30 a.m.	1/16 in. immediately; 1/32 in. next half hour.	No settlement thereafter.
Oct. 28, 1914	40.0	8:38 a.m.	4:25 p.m.	5/64 in. immediately.	No settlement thereafter.
Oct. 28, 1914	40.5	4:49 p.m.	7:15 a.m.	1/64 in. immediately.	No settlement thereafter.
Oct. 29, 1914	45.0	8:00 a.m.	8:00 p.m.	1/32 in. immediately; 1/32 in. next half-hour; 1/32 in. between 12:30 and 1:30 p.m.	No settlement thereafter to time of load off.
Oct. 29, 1914	....	.....	.....	Up 1/32 in. when load went off. Total, 9/32 in., except initial 3/16 in.	

first being taken to place about a foot of broken stone in the bottom of the pile, and to tamp it well. After filling, each pile was given about five blows of the driver to settle it in place. Concrete was then placed around the pile tops to a depth of about a foot above them, and a double grillage of I-beams was placed to distribute the load over the piles. The foundation concrete was then brought up to within 2 in. of the bottom of the cantilever grillage girders, in readiness for the steelwork.

It took just one month to place the 23 piles, working 24 hr. a day and 7 days a week.

For the foundation for the end columns, on both the east and the west ends, the concrete was placed on a bed of broken stone, which had been tamped solidly to a depth of 1 ft. In one operation the concrete was brought up to a level 1 in. below the bottom of the grillage beams. The steel erection was straight work and presented no unusual difficulties.

⁂

. **The Seacoast Fortifications** of the United States are described in the annual report of the Chief of Engineers. The present seacoast defenses were planned by the Endicott Board of 1886 and the National Coast Defense Board, of which Hon. Wm. H. Taft was president, which reported in 1906. Fortifications in accordance with the recommendations of these boards have been built at 26 different seaports; in addition, work is in progress on coast defenses at Los Angeles, and batteries are shortly to be built at the entrance to Chesapeake Bay. The total amount appropriated to date for building gun and mortar batteries is a little less than $30,000,000, and a little over $1,000,000 has been appropriated for modernizing some of the older batteries. Additional expenditures for fire control at fortifications, electrical installations, searchlights, preservation and repair of fortifications, seawalls and embankments, and the purchase of sites for fortifications amount to about $15,000,000 additional. The Chief of Engineers says: "When designed and constructed, our seacoast batteries were thoroughly modern and fully adequate for the purpose for which they were intended; but the work of battery construction has in the past few years been allowed to practically cease and has not kept pace with the recent progress in naval development. Unless our fortifications are to become obsolete, changes in any of the elements of naval defense must be met by corresponding changes in seacoast batteries, and these changes must in most cases consist in the construction of absolutely new batteries in new locations. If the guns mounted in the older batteries had been of longer range, the locations of the batteries would in many cases have been different."

Reinforced-Concrete Coal Pier for Pennsylvania R.R.

In the construction of its new coal pier at Canton wharves, Baltimore, Md., the Pennsylvania R.R. is utilizing a type of reinforced-concrete pier that showed a notable saving over the timber and earth-fill design first proposed. The pier, which is to be 66 ft. wide and 942 ft. long, is to be equipped with the most modern coal-handling machinery. The usual barney cars propel the coal cars up an incline to a car dumper that empties the cars into a hopper, from which small cars deliver the coal through conveyors into the holds of vessels lying alongside the pier. The plant provides facilities for loading 20,000 tons of coal per day to vessels and is to be approached through a yard with a capacity of 1,000 cars.

As at first designed, the pier consisted of a substructure of timber piles carrying a timber floor on which were to be an earth-fill about 4 ft. deep and four longitudinal concrete stringers to carry the steelwork of the superstructure. A section through this pier is shown in the upper part of the accompanying drawing. As an improvement on this design, the company later adopted the one shown in the lower part of the drawing, which is now being executed.

The adopted design is entirely of concrete above low water. It consists of four longitudinal massive concrete stringers carried on timber-pile bents (with inclined piles, as shown) and carrying a reinforced-concrete floor made up of transverse beams 12 ft. 6 in. c. to c., with an integral 10½-in. floor. The superstructure reactions come above the stringers.

To show the actual saving in weight of the reinforced-concrete pier over a timber-floor earth-fill, the quantities in the accompanying tables, taken from the engineers' figures, are of interest.

TABLE 1. WEIGHT OF TIMBER-FLOORED, EARTH-FILLED PIER (FIG. 1)

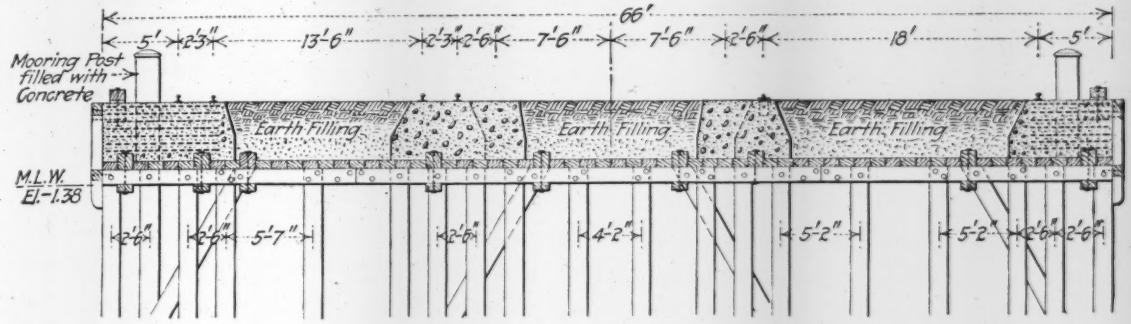
	Tons
686,000 ft. b.m. yellow-pine wet timber at 4½ lb.	1,543
3,530 cu.yd. concrete at 3,900 lb.	6,883
Reinforcing steel	19
5,630 cu.yd. earth-fill at 3,000 lb.	8,445
Bolts and cast-iron washers	34
Total	16,924

TABLE 2. REINFORCED-CONCRETE PIER

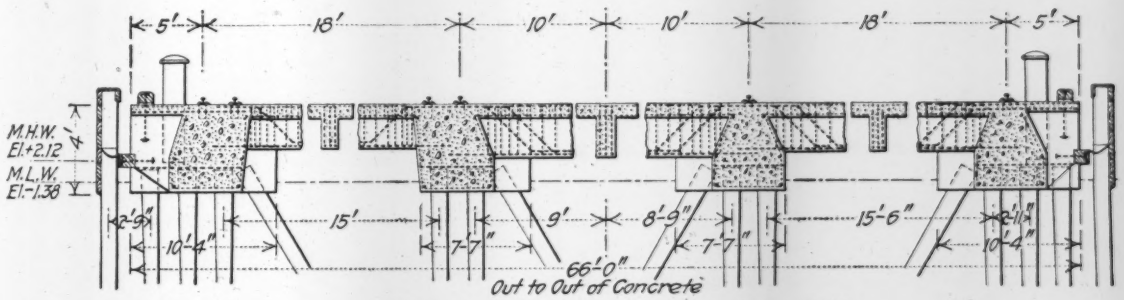
	Tons
5,960 cu.yd. concrete at 3,900 lb.	11,622
Reinforcing steel	118
Bolts and washers	2
50,000 ft. b.m. timber waling strips, etc., at 4 lb.	100
Total	11,842

The difference of over 5,000 tons in the weight of a dock is illustrated in the comparative number of piles required—for the timber dock, 4,332; for the reinforced-concrete dock, over 1,300 less piles, or 3,084.

The coal pier is being built by the Pennsylvania R.R., Alexander Shand, Chief Engineer. The Cleveland Dock Engineering Co., Cleveland, Ohio, designed the pier substructure, which is being built by the Arthur McMullen Co., of New York. The dredging has been done by the Maryland Dredging and Contracting Co., of Baltimore, Md., and the superstructure will be furnished by the Mead-Morrison Manufacturing Co., of Chicago.



Proposed Timber and Earth Fill Design



Reinforced-Concrete Design being Built

PROPOSED AND ACCEPTED CROSS-SECTIONS OF COAL PIER FOR PENNSYLVANIA R.R. AT CANTON, MD.

Electric Cooking and Heating are being stimulated by the Potomac Electric Power Co., of Washington, D. C., which has made a radical reduction in the rates covering current used for such purposes. The rates, effective Dec. 25, 1915, are 10c. per kw.-hr. for the first 10 kw.-hr. used each month, then 3c. for the remaining current used during the month. In order to separate the current used for heating and cooking from that which may be used for lighting, a separate meter will be installed for each user, the minimum charge for such meter being $1 per month unless there is a lighting meter on the same premises, in which case the charge is 50c. It is estimated by the company that the charge to the consumer will be practically the same as the charge for gas for corresponding service.

Oiling Illinois Earth Roads

The economical maintenance of good earth roads in the black-soil district of Illinois is a specially difficult problem, an effective solution of which seems to be the oiling of such roads. Oiled earth roads have been used in Iroquois County with good results, although the effect of the winter is yet to be seen. Several townships last summer oiled an aggregate of about 20 mi. of roads in the heavy black-soil districts, no work being done on the sand roads. In a paper presented at the annual meeting of the Illinois Society of Engineers and Surveyors by Benjamin Jordan, county superintendent of highways, it is stated that even in the wettest weather, when other roads were almost impassable for machines, these oiled roads were practically as good as asphalt paving.

One township used for 8 mi. of road three tank cars of oil (30,441 gal.), costing $829, which was paid for out of the township-road fund. Farmers donated teams to distribute the oil, so that the 8 mi. cost little over $100 per mi. The result is that three townships will vote on a proposition to levy a special tax of 40 to 50c. per $100 for a period of five years, to be expended in oiling the roads. The vote is to be taken in April, so that the people will be able to see how the present oiled roads stand the winter service.

Each of these townships is supplied with tractors, road graders, etc., and one has purchased a 1,000-gal. oil distributor. The plan is to coöperate so that when one township receives a shipment of oil, it will have the use of the entire plant of the three townships for unloading the tank cars rapidly and distributing the oil economically.

In another paper on the same subject, presented at the meeting by B. H. Piepmeier, maintenance engineer of the Illinois State Highway Department, it was pointed out that the main purpose of oiling earth roads is to suppress the dust and aid in maintaining a smooth and waterproof surface. The oiling does not make a permanent road, but the average user of earth roads is willing to bear three or four months of bad roads during the winter if he has assurance of reasonably good roads for the remainder of the year. Some people think that after a road has been oiled systematically for a number of years the surface will become saturated and no further oiling will be necessary. Examination of streets and roads that have been oiled annually for ten years shows that this idea is erroneous, and taxpayers should be warned against such expectations. While a little less oil may be required after several years' treatment, the oiling cannot be eliminated if a good road is to be maintained.

There are, however, many earth roads that can be maintained effectively by the intelligent use of oil and the road drag. It must not be expected that this treatment of a main road will make it satisfactory for heavy traffic. The surface should be properly graded, drained and rolled before oiling, but this work should not be charged to the cost of oiling. It is better to oil on a wet surface than on a dusty surface, but the road should not be so wet that the wheels of the distributor cart will pick up mud.

For the first application ½ gal. of oil per square yard may be required, or ¼ gal. if the road has been oiled before. It is better to apply small quantities frequently than to make one heavy application, and one of the best roads of this kind was treated four times during the summer at the rate of ⅛ gal. per sq.yd. for each treatment. Dragging an oiled road is absolutely necessary for maintaining it in good condition.

A light application of sand is a justifiable expense for a majority of oiled earth roads. It retains the oil and assists in preventing wear and keeping down the dust. It may be applied by shoveling from a wagon or by a distributing apparatus, in the proportion of 1 cu.yd. of sand to each 150 sq.yd. of road.

Novel Water-Treatment Plant at Excelsior Springs, Mo.

The water-supply of Excelsior Springs, Mo., is obtained from driven wells in bottom lands of the Missouri River at Miltondale, about 7 mi. from the city. The city contains about 5,000 permanent residents and some 3,000 transients per year. There are 720 service connections, all metered.

The water from the wells contains a considerable amount of iron and at times is highly colored and bad tasting. An analysis shows 10.7 p.p.m. of iron, 72.8 parts of carbon dioxide and an alkalinity of 365 parts. The extraordinary amount of CO_2 is noticeable in the effect on lead fixtures and plumbing.

To eliminate these objectionable features, the Excelsior Water, Gas and Electric Co. (S. W. Henderson, vice-president and general manager) employed the firm of Black & Veatch, consulting engineers, Kansas City, Mo., to design and supervise the construction of the apparatus described in this article. The plant, which was completed and tested in August, 1915, consists of an aërating tower of unusual design, settling basins, mixing chambers, mechanical filters and a clear-water storage well, from which the water is pumped into the city mains. The capacity of the filter plant is 1,000,000 gal. daily.

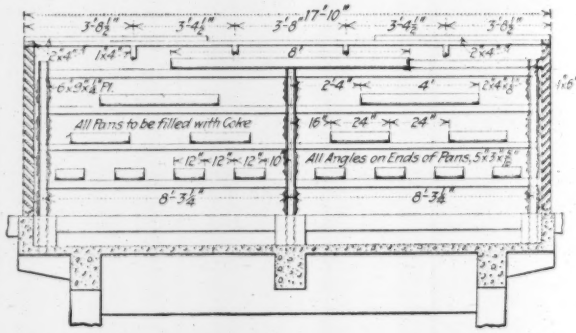


FIG. 1. CROSS-SECTION, SHOWING TYPE OF WATER AERATOR, EXCELSIOR SPRINGS, MO.

The pans are filled with coke. The water flows over the sides of each pan. The vertical drops are 1.9 ft. from the first and 2.3 ft. from the bottom set of pans and the pans are 20 ft. long. The drawing here reproduced is a section of an aërator built at Emporia, Kan., but the cross-sectional dimensions of the two plants are substantially the same, except that the vertical drops in the Emporia plant are 1.25 ft. instead of as stated above for the Excelsior Springs plant

An analysis of the raw water collected on Aug. 30, 1915, by F. M. Veatch was as follows, the quantities being parts per million: Total solids, 527; SiO_2, 3; $Fe_2O_3 + Al_2O_3$, 56.6; Ca, 101.2; Mg, 16.82; Na + K, 6.2; Cl, 8; SO_4, trace; HCO_3, 420.

The water is colorless when first pumped from the ground, but becomes a yellowish red on exposure to the air. The aërator (Figs. 1 and 2) is the most interesting feature of the plant. After being aërated, the water passes through a concrete flume to the mixing chamber, where milk of lime is added. The water then goes to two settling basins (Fig. 3), either one or both of which may be used.

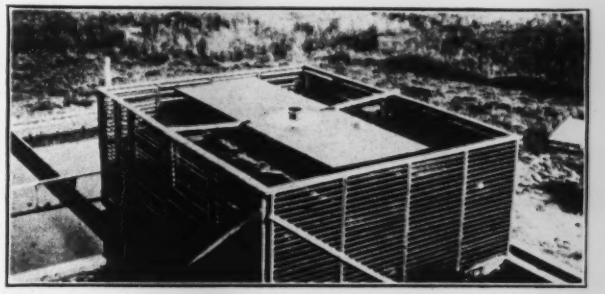


FIG. 2. VIEW OF WATER AERATOR, EXCELSIOR SPRINGS, MO.

Results of Tests of Operation

The accompanying table gives the results of tests made in the latter part of August by C. C. Young, chemist of the Kansas State Water Survey Laboratory; F. R. Hesser, assistant engineer, Kansas State Board of Health; and F. M. Veatch, of Black & Veatch. The tests were made after the plant had been in operation about 10 days. What follows is based on a report of these tests.

The water had a cloudy yellow appearance after the addition of the lime. All color was removed by the filters. No floc was observable in the basins or on the filters. The raw water had a carbon dioxide content of 85 p.p.m. After passing the aërator this was reduced to 27 p.p.m., and after the lime treatment on reaching the end of basin No. 2 the CO_2 had been reduced to 19 p.p.m. In the filtered water it was 11 p.p.m.

There was no increase in bicarbonate alkalinity by the addition of the lime. This was an unusual condition in so in this respect the aërator is giving entire satisfaction. The determinations (in the table) showing 106 and 103% dissolved oxygen are probably due to the presence of algæ, which were growing in the reservoir. The average reduction of carbon dioxide by the aërator was 59% of the total, of which 24% was removed in flowing over the first pan, 6.27% by the second, 3.9% by the third, 16.7% by the fourth pan and 8.1% by the fall into the collector.

Under ordinary weather conditions this leaves about 30 p.p.m. of CO_2 in the water, which is sufficient to prevent a large part of the iron from precipitating. At the beginning of the tests sufficient lime was not added to neutralize the CO_2, which accounts for the fact that

REDUCTION OF IRON AND CARBON DIOXIDE BY WATER-TREATMENT PLANT, EXCELSIOR SPRINGS, MO.

	Parts per Million											Lime Used			
Test For	Raw Water	Pan No. 1	Pan No. 2	Pan No. 3	Pan No. 4	Collector	Basin No. 1 Middle	Basin No. 2, Middle	Basin No. 1, Second Bay	Basin No. 1, Fifth Bay	On Filter	Filtered	% Sol.	Gr. per Gal.	Gal. Filtered per Min.	Remarks
Total Iron	8	..	..	..	..	..	2.5	...	...	...	2.0	0.2	1	1.1	512	Both basins in use
	12	..	..	..	..	..	2.5	...	...	...	1.3	0.3	1	1.1	512	Normal operation
	16	..	..	..	..	12	...	3.5	...	...	3.5	0.2	1	1.1	512	
	10	..	..	..	..	9	...	...	...	...	0.2	0.1	1	3.8	512	
	10	..	..	..	..	10	...	...	0.2	0.2	0.2	0.1	2	6.9	512	Baffled basin used
	9	..	..	..	..	9	...	...	0.2	0.2	0.2	0.1	3	7.9	512	Second basin cut out
	10	..	..	..	..	9	...	...	...	0.2	0.2	0.1	3	7.9	512	
Dissolved Oxygen	0	23	25	49	78	..	106	103	...	...	.....	9.3	1	1.1	...	Figures represent % saturation for temperature of collection
	0	26	47	56	68	86	80	80	...	...	.....	9.3	..	1.1	...	
	0	25	48	63	62	88	80	80	...	...	.....	8.6	..	...	...	
Carbon Dioxide	86	59	51	52	35	27	21	19	...	...	19	11	1	1.1	512	Normal operation
	74	55	54	55	36	30	...	...	8	...	11	13	2	3.8	512	
	80	60	60	56	45	46	...	...	10	...	5	.....	2	6.9	512	One basin in use; non-baffled basin cut out
	63	51	42	36	29	21	...	...	4	...	2	2	3	7.9	512	
	61	52	47	41	33	25	...	...	2	...	1.5	2	3	7.9	512	
Bicarbonate Alkalinity	356	..	..	..	..	..	...	...	...	...	356	354	..	1.1	512	Normal operation
	360	..	..	..	..	..	...	...	...	...	362	358	..	1.1	512	
	366	..	..	..	..	..	...	...	...	...	346	.....	..	3.8	512	
	364	..	..	..	..	..	...	...	...	...	.....	350	..	6.9	512	
	374	..	..	..	..	..	...	...	...	...	304	300	..	6.9	512	
	372	..	..	..	..	..	...	...	...	...	276	272	..	7.9	512	

view of the fact that as much as 7.9 gr. of lime per gallon was used in some of the tests. The normal carbonates (CO_3) did not appear, and the bicarbonate alkalinity was reduced 100 p.p.m. while the 23.5 p.p.m. of CO_2 was being neutralized. Apparently the series of reactions $2\ CO_3 + Ca(OH)_2 = Ca(HCO_3)_2$ and $Ca(HCO_3)_2 + Ca(OH)_2 = 2\ CaCO_3 + H_2O$ was not followed at any time during the tests.

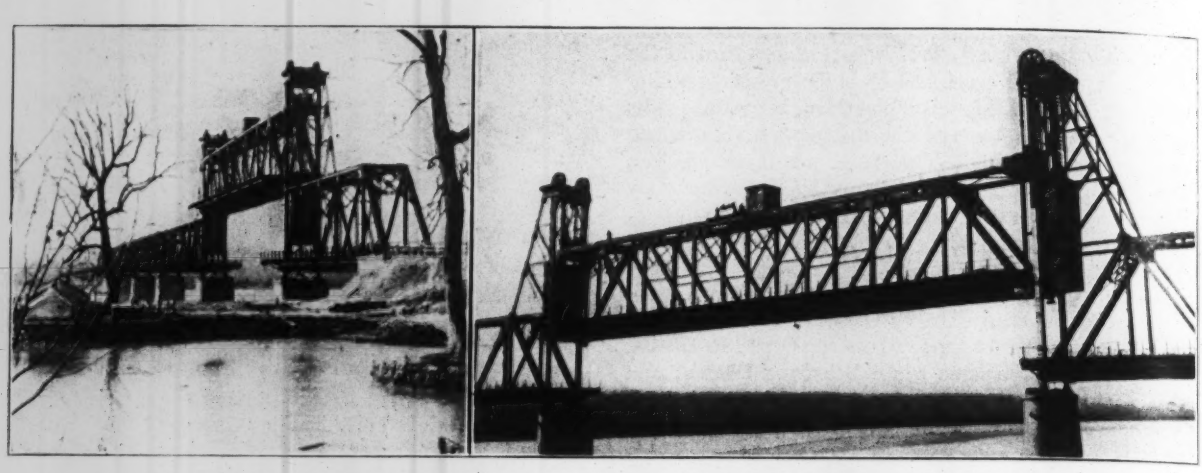
Action of the Aërating Device

The raw water contained approximately 10 p.p.m. of iron, which after the addition of the lime was reduced to 2 p.p.m. and after filtering to 0.2 p.p.m. The total amount of iron reduction guaranteed was 0.5 p.p.m. The iron is in the water in solution in an unoxidized state and will not precipitate until oxidized and until the greater part of the carbon dioxide has been removed. The twofold object of the aërator is therefore to oxidize the iron and to allow the carbon dioxide to escape.

Under ordinary climatic conditions the water becomes saturated with oxygen by the time it reaches the collector, some of the iron was carried over into the filters. After the preliminary tests were made, the second basin was cut out to increase the rate of flow, and the amount of lime was increased.

The effects of increasing the lime from 1.1 to 3 gr. per gal. were that the flocculent precipitate of iron hydroxide and calcium carbonate appeared at once in the mixing chambers; the deposition of the precipitate took place largely in the first bay of the settling basin, and at the end of 5 hr. the remainder of the basin was practically clear and the water on the filters had only a very slight turbidity. The carbon dioxide in the water on the filters was reduced to 11 p.p.m. and the iron to 0.2 p.p.m. The filtered water contained less than 0.1 p.p.m. of iron.

The tests were continued with 6 and with 7.8 gr. of lime per gallon, in the hope of effecting a complete removal of the CO_2, but without success. In accordance with the recommendation of Mr. Young the aërator has since been operated with the first three pans filled with Connellsville egg coke. The lime dose is 2.6 gr. per gal.



FIGS. 1 AND 2. LIFT SPAN HUNG BY CHAINS; ARKANSAS RIVER BRIDGE, PINE BLUFF, ARK.
By shifting towers any span can be made the lift span to suit channel location

Pine Bluff Vertical-Lift Span Suspended by Chains

SYNOPSIS—Chains used in place of wire ropes to suspend 239-ft. vertical-lift span across Arkansas River. "Folding" counterweight to balance the chains is another new feature. Various details of novel design shown by drawings.

Hung by chains instead of by ropes, the lift span of a new county bridge across the Arkansas River at Pine Bluff, Ark., represents a new departure in draw-bridges. Various other features of the lift span are original. The engineers, Hedrick & Cochrane, of Kansas City, boldly disregarded precedent in many ways, not merely in the machinery, but also in the structure. The two wagonways, for example, are placed on long brackets outside the trusses, the deck between trusses being used for electric-railway tracks.

The main low-water width of the river is nearly 2,000 ft. The channel may shift to any position in this distance, as happened at the near-by Cotton Belt bridge, which now has two swing-spans, one near each bank, the channel having shifted from one bank to the other. To provide for such a contingency the county bridge is so arranged that any one of its spans may be made the drawspan. The use of a vertical-lift span enabled this plan to be carried out easily; lift mechanism and tower are so built that they can be shifted to any one of the channel spans, and thus any span—six are of equal span-length—may be converted into the lift span. This expedient has been used before and is known to involve but little extra cost.

The lift span is hung from steel chains instead of from ropes. Also, instead of a lifting cable a lifting chain running over sprockets is used.

The great weight of the counterweight chains making it important to compensate for change of balance due to shifting of the chain weight as the span rises, an expanding counterweight was used as an auxiliary to the main counterweight. It is picked up by the main counterweight as the span goes down, and released as the span goes up.

The six channel spans are each 239 ft. 4 in. long, center to center of end pins. These, as well as a north approach span of 149 ft. 7 in., are of full riveted type, with 29 ft. 11 in. uniform panel length. The channel spans allow for a clear opening of 225 ft. The one adjoining the short span is the present lift span.

The piers were all sunk by pneumatic caissons to 70 ft. below low water, bringing up on sand and coarse gravel.

Bronze-Bushed Pin-and-Link Suspension Chains

In the cable counterweight arrangement used in other bridges, a series of wire ropes at each of the four corners of the span is attached in successive pairs of ropes by means of sockets to a number of superimposed equalizer levers, which gradually bring the pull of a number of ropes to a single point of attachment. By using link chains in the present bridge the complication of multiple equalizers is avoided.

The chains consist alternately of four and five links, all of them steel forgings, joined by 3½-in. pins (Fig. 4). Provision for the turning of links on pins was made in alternate links only, the intermediate links being made narrow (with higher pin pressure) to save width of chain and therefore length of pin. The wide links, which turn on the pins, are fitted with phosphor-bronze bushings to reduce friction (see Fig. 4); the bearing pressure is 6,000 lb. per sq.in. projected area. The nonrotatable links are narrow and give a pin pressure nearly three times as great. To hold the pins from turning in these links there is a 1¼-in. diametral hole at each end of the pin just outside the outer link, and a keeper or key bar passed through this hole is bolted to the link by a ⅝-in. bolt.

No provision was made for lubrication. It was thought that, the motion being so slow and so infrequent, lubrication would not be necessary.

When the span was put in operation, many of the key bars bent and some of them broke, showing that the wide phosphor-bronze bearings gripped the pin more tightly

than the thin unbushed "fixed" links. Victor H. Cochrane, of the consulting engineers, says that the cause has not been definitely determined, but that it is believed that the bronze bushings had been machined to a tighter fit on the pin than the pin links. A contributing factor in the trouble may have been temperature expansion, as most of the trouble developed on the approach of warm weather; the greater expansion of bronze than of steel may have helped to cause the bushings to seize on the pin.

For temporary repair, the bent or broken key bars were removed and simple bolts put in their places to act as cotters and keep the links on the pins, irrespective of which set of links rotated. Subsequently, harder key bars were put in than the original medium-steel keys.

The main counterweights of the bridge are of concrete—precast blocks supported in a steel frame. These counterweights are detailed in such a way that they can be taken apart and built together easily when changing to some other span as lift span.

The shifting weight of the counterweight chains is balanced by a set of expanding counterweights (Fig. 1) made of cast-iron disks hung in a loose string from the top of the tower in such arrangement that the main counterweight picks the weights up one by one as the span goes down, and thus constantly increases the counterweighting as the amount of chain on the channel side of the tower increases in the descent of the span. The disks are each 3 ft. 5 in. in diameter and 2¼ in. thick, hung by two 1¾-in. bolts passing through lugs at diametrically opposite points. Successive disks have their suspension bolts in successively different angular position, so that the bolts do not interfere when the counterweight folds up. In the expanded position the disks are 1 ft. apart in the clear. The engineers had in mind the possibility of producing excess compensation in this folding arrangement, by making the disks slightly heavier; such overbalance would help in starting the span from rest and in checking it at the end of its travel. This arrangement was not used, however, but the expanding counterweight was proportioned to balance the chain weight exactly.

Two vertically arranged sprocket chains along each tower post, eight in all, fitted with equalizers and having adjustment to regulate the tension, furnish the means for moving the span up or down (Fig. 7). Just above the top chord of the lift span each pair of sprocket chains is

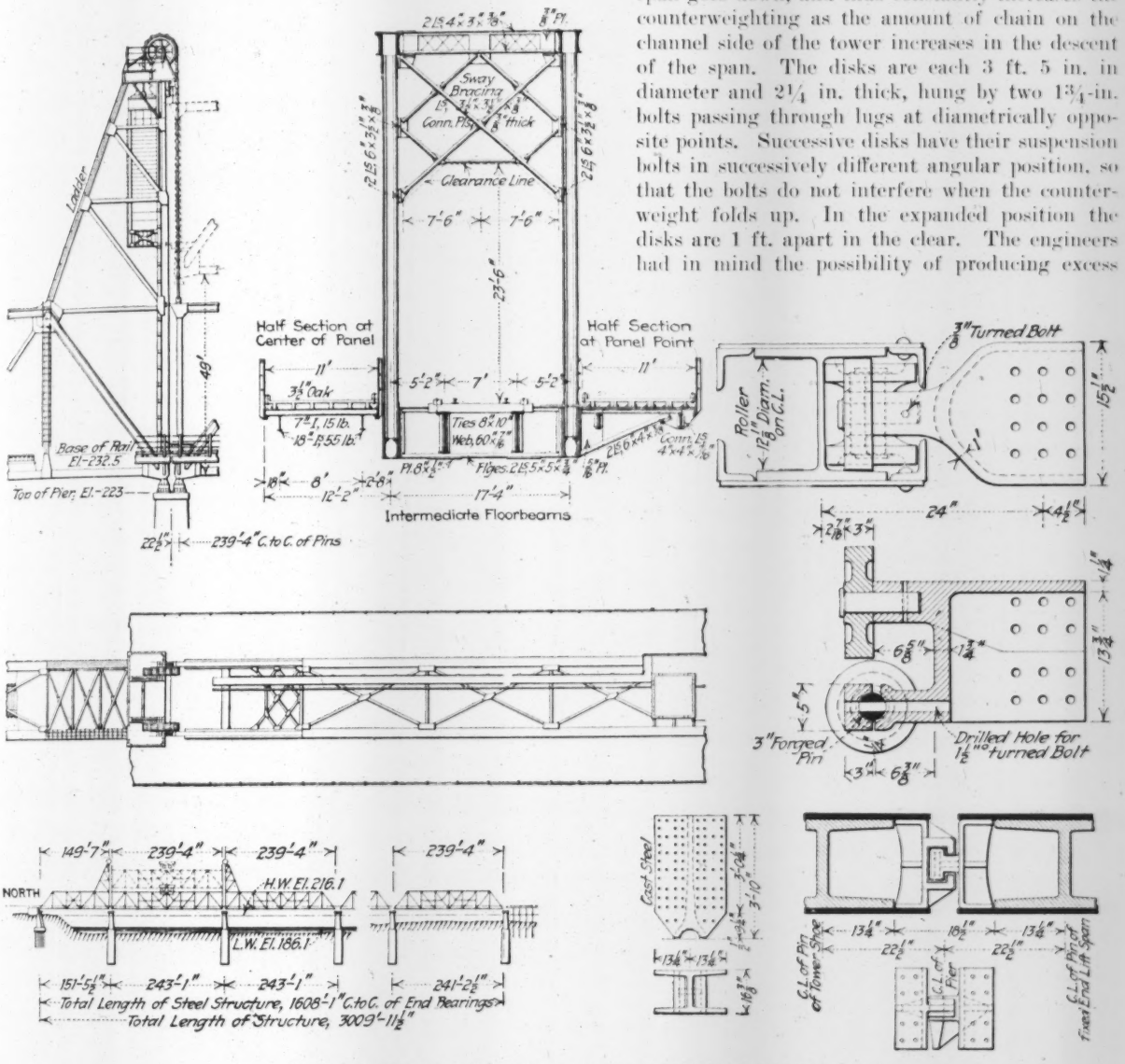


FIG. 3. PINE BLUFF BRIDGE OVER ARKANSAS RIVER

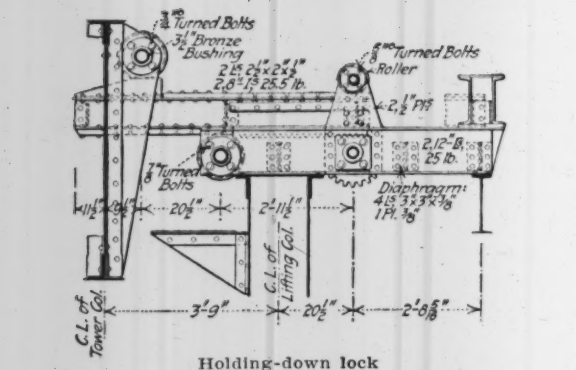
General elevation and section; bearing casting at end pin; guide rollers and traction lock of lift span

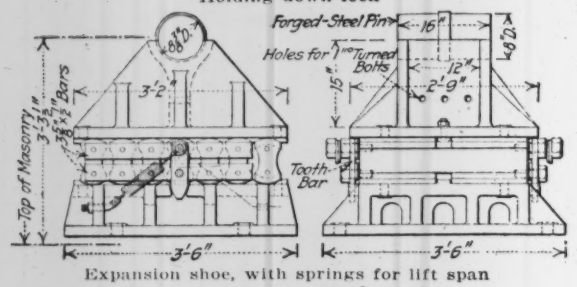
looped back toward midspan between two pairs of idler sprockets and around driving sprockets. The latter are operated by a motor through gearing. One motor at each end of the span drives the two sets of sprockets. A longitudinal shaft connecting the gearing at the two ends of the span insures equality of motion at the two ends. It also provides against failure of one of the motors, each motor being capable of operating the span alone. The hand-operating capstan is attached to the longitudinal shaft at midspan.

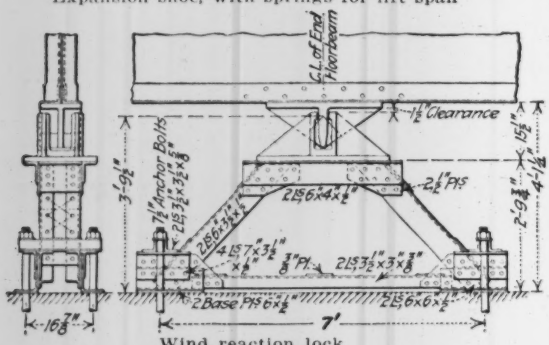
The motors are 1,200-volt d.-c. motors, operated by 600-volt current. It was intended to operate them from the current of the proposed electric railway, but as the railway was not built they had to be supplied by a 2,300-volt a.-c. transmission line. The only available motor-generator set converted to 600 volts, and the motors are operated at this pressure, with no particular difficulty. The rated power at 1,200 volts is 40 hp. per motor.

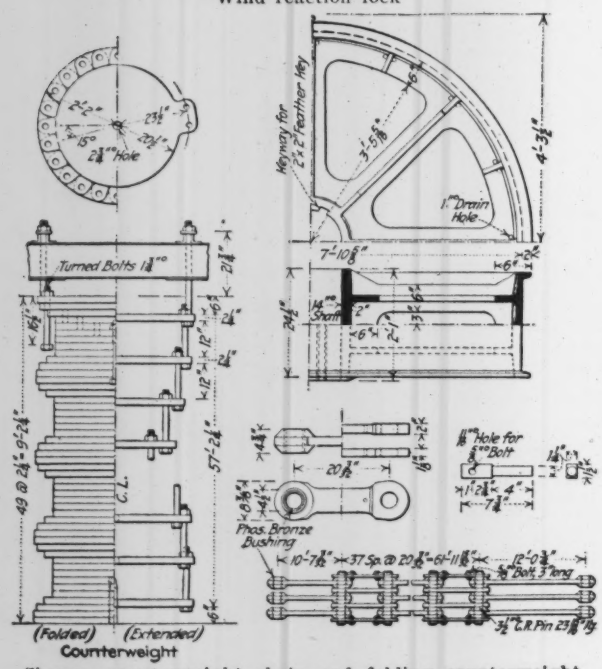
Wedge Locking Bars at Ends of Span

A detail in Fig. 4 shows the end locks. These are long bolts moved by rack and pinion through a small motor. The upper side of the locking bolt is beveled 4









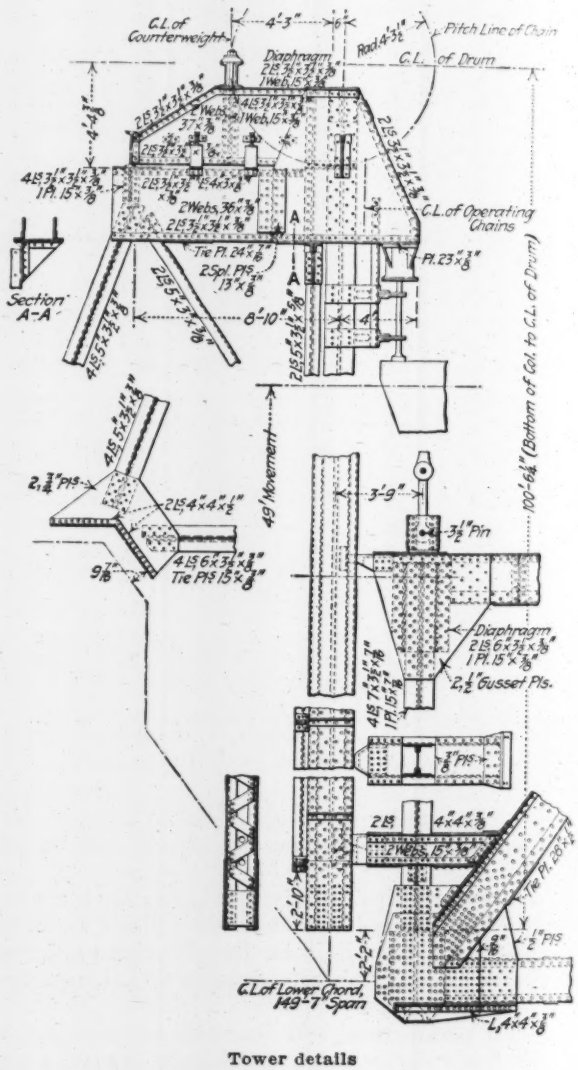


FIG. 4. IMPORTANT DETAILS OF PINE BLUFF BRIDGE LIFT SPAN

in. in 18 in., so as to force the span down to seat in case it should not be down.

The lift span is guided by the web and flanges of the 14-in. I-section of the tower posts. Guide rollers (Fig. 3) attached to steel castings on the longitudinal strut that continues the line of the top chord from the hip to the counterweight hanger run in the post channel-ways. At one end of the span there are two longitudinal rollers 10 in. in diameter and a transverse roller 12½ in. in diameter, the latter being beveled to suit the bevel of the flange of the I-section. At this end therefore the span is guided both longitudinally and transversely. At the opposite end the two longitudinal rollers are omitted, so that the span is free to expand and contract longitudinally.

These rollers, however, do not have to transfer the traction stresses of braked trains. These are absorbed by a tongue-and-notch engagement of castings riveted between the gusset plates at the fixed end (Fig. 3).

For lateral centering and to transfer the wind stress there is at each end of the span a casting attached to the end floorbeam of the lift span, so arranged that a tongue on this casting engages a recess in another casting carried on the steel frame attached to the pier (see Fig. 4).

FIG. 5. LIFTING CHAINS AND EXPANDING COUNTERWEIGHT, FROM MIDDLE OF LIFT SPAN

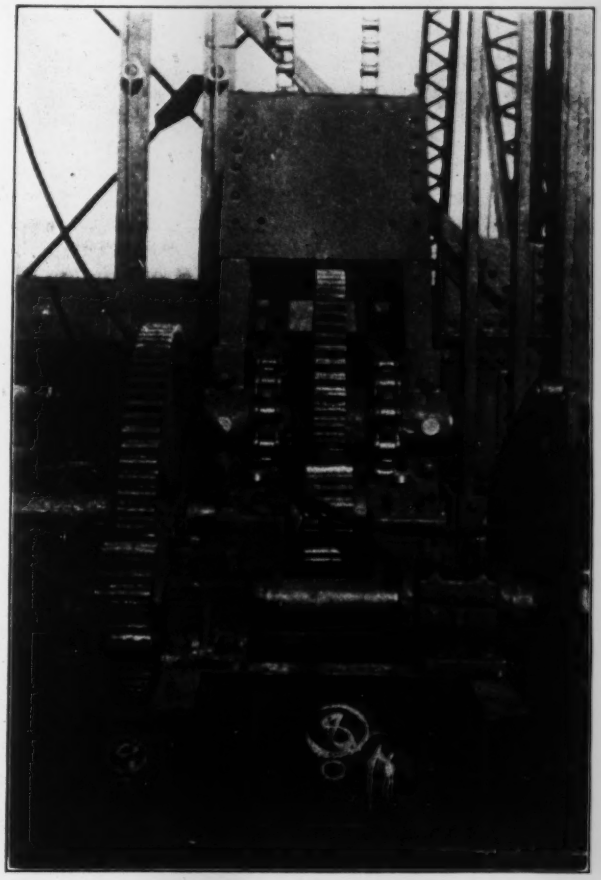


FIG. 6. DRIVE CHAINS FOR RAISING LIFT SPAN

The rail joints are made by a casting each on span and pier. The castings on the lift span have a ¼-in. side bevel to prevent striking the fixed rail as the span comes down to seat.

The shoes of all the spans are designed to be capable of doing service as shoes for the towers. The shoes on which the towers are seated are fixed; that is, they have no rollers. All the expansion bearings of the fixed spans, however, have rollers. In consequence, castings are provided to be inserted in place of the rollers to convert an expansion bearing into a fixed bearing when it is to carry the tower.

As the rollers under the lift span are free when the span is open, they can become displaced. To prevent this, they are held in vertical position by a pair of springs attached to a tongue bar of the middle roller as shown in Fig. 4. The lateral-centering bar used on the other shoes, engaging grooves at midlength of the rollers, is omitted in the case of the lift span. In all other respects the lift-span shoes are the same as used with the fixed spans.

The upper pin casting of each span is a deep casting riveted between the webs of the end post, adapted to take and transfer the large load from the tower. This casting is shown in Fig. 3, and its position in Fig. 4. Since the tower leg must carry the weight of half the span plus its share of the counterweight, or double the normal load, the shoes are a little more than twice as heavy as they would be in ordinary bridge service.

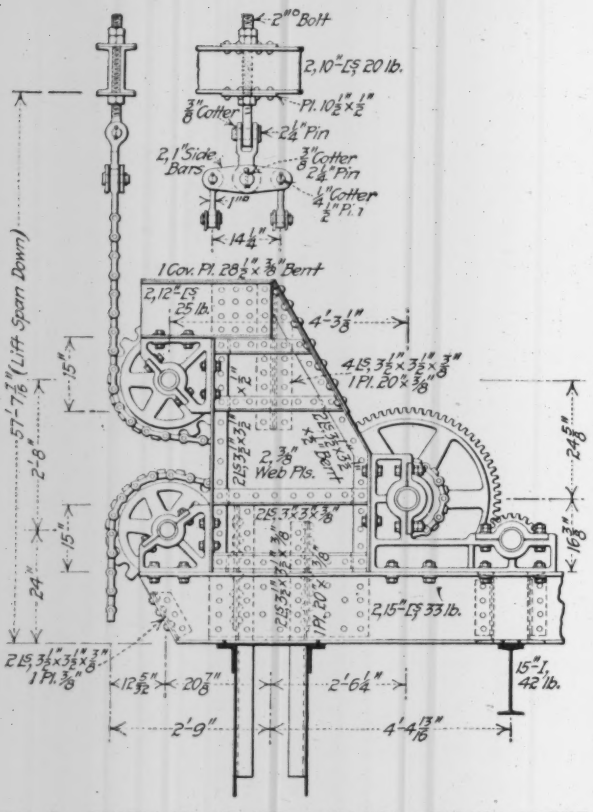


FIG. 7. DRIVE CHAIN AND GEARING OF PINE BLUFF LIFT SPAN

The bridge was built by the Missouri Valley Bridge and Iron Co., Leavenworth, Kan., for whom the American Bridge Co. furnished the steel. Hedrick & Cochrane, the consulting engineers, were represented in the field by C. S. Reagan.

A New Concrete Pile Driven from the Point

By C. S. Howell*

Concrete piles of a new type are being driven in building the foundations of the Manhattan State Hospital power house on Ward's Island, New York City. In driving these 105 precast piles to bedrock at depths varying from 12 to 20 ft. below the footings, a follower consisting of a pair of driving frames resting on the shoulder of the iron point of the pile and extending beyond its head was placed around the pile, as shown in Fig. 1.

The piles are spaced on average centers of 3 ft. The soil is a very compact sandy clay, and many boulders and smaller stones exist at various depths, mostly just above bedrock. Footing trenches will be excavated after the piles have been driven, which will involve from 7 to 10 ft. of additional penetration above the head of each pile.

The piles are 16 in. square, with corners beveled 1 in. to prevent chipping in handling. They are made of 1:2:4 concrete, ¾-in. crushed traprock and graded sand being used for aggregates. The mix is made wet to pour into the forms. Casting is done in stacks three tiers in height. Each pile has four twisted steel rods extending the full length, and 3 ft. beyond the head for anchorage in the footings. The rods are ⅝- or ¾-in., according to the length of the pile. No tie wires are used. The method of casting provides for holding the rods in correct position while pouring.

After casting, the piles are left to season for four weeks before driving. Forms are removed in 48 hr. and reused. Casting of these piles was carried on from Nov. 22, 1915, to the latter part of December. After seasoning for two weeks, the piles were handled with the same freedom that would attend the handling of timber piles.

The most distinctive feature about these piles is the cast-iron point, which is 19 in. square and weighs over 300 lb. It is hollow, except for four diagonal walls, or webs, through which the reinforcing bars are drawn and bent back upon themselves, the bars being heated for this purpose in a forge on the work. The point has at the top a collar that fits into one end of the casting form, and the rods are drawn through the opposite end of the form, having first been anchored in the point.

The method of driving involves a wide departure from existing practice. Reference to the accompanying drawings will illustrate this in detail. While the practice of driving by means of an inclosing steel casing is not new, it will be noticed that this method uses two frames, on opposite sides of the pile and resting on the iron point. These are of channel shape and have at the bottom two projections that fit into slots cast on corresponding opposite sides of the point, to hold them in position upon the extended part of the point while driving. Earth pressure also serves the same purpose as these projections and their corresponding slots, after sufficient penetration is obtained. The object of using two driving frames is to overcome resistance against pulling them from the

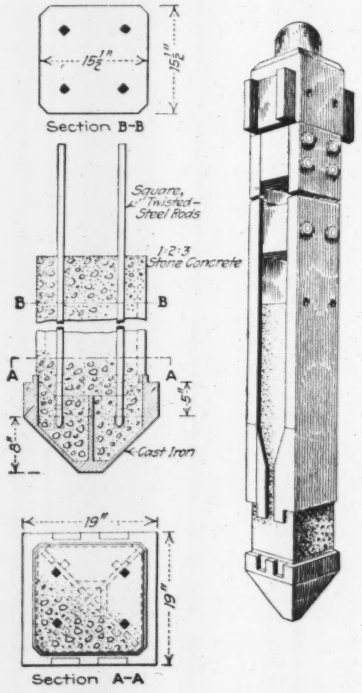


FIG. 1. THE "GIANT" CONCRETE PILE WITH DRIVING CASING ATTACHED, AND DESIGN DETAILS OF THE PILE

*Chief Engineer and General Manager, Giant Concrete Pile Co., 37 East 28th St., New York City.

ground after the desired penetration is reached and to provide sufficient earth friction against the pile to prevent its being disturbed when the casing is pulled.

By this method of driving, the reinforcing rods are made to transmit the resulting frictional stress to the concrete and to distribute this stress over the full length of the pile. A 5,000-lb. steam hammer (total weight, 10,000 lb.), with normal stroke of 42 in., is used to drive these piles. The weight of the hammer rests directly upon a steel-bound oak block held in a steel casting, to which the two driving frames are rigidly secured at the top. The oak block extends through the hammer base and is capped with a steel plate, to receive the hammer.

FIG. 2. A NUMBER OF REINFORCED-CONCRETE PILES WITH CAST-IRON HEADS ATTACHED, READY FOR DRIVING

The driving frames are considerably longer than the pile, the space above the pile head varying according to the length of the pile. At the head of the pile a wood block may or may not be bolted between the frames (bored for the reinforcement), to take a part of the stress due to earth friction on the two partly exposed sides of the pile, when the depth of penetration develops a pull in the rods in excess of their safe tensile strength. Thus far such a block has not been used on the hospital work. Since the point is driven by the frames and inasmuch as these frames also relieve the pile of all earth friction, except as already noted, the pile is actually made to occupy the displacement of the point, as penetration proceeds, by means of the rods.

The first of these piles was driven on Dec. 30, 1915, to a depth of 17 ft., when refusal under the last 30 blows indicated bedrock and closely checked the preliminary borings. The pile was driven in 13 min., 290 blows being required. The rapid penetration, though it proceeded in constantly decreasing increments, was due to the weight of the driving apparatus, there being over 22,000 lb. in movement. The second pile was driven 6 in. deeper and required 336 blows and 15 min. The piles were not damaged in any manner in driving, there being no chipping or indication of cracks.

Municipal Street Sprinkling

The charter of the City of Rochester, Minn., was amended early in 1915 to allow the city to do its own street sprinkling, charging to the frontage the exact cost of the work. The results were described by J. C. Utton, City Engineer, in a paper presented at the annual meeting of the Minnesota Surveyors and Engineers Society at St. Paul.

The district comprised 5¾ mi. of paved streets and 4 mi. of unpaved streets. The city had four 600-gal. sprinkling wagons, and the district was divided into four sections to equalize the travel of the teams, which averaged about 14½ mi. daily. The paved streets were sprinkled four times daily and twice on Sunday, and the others were sprinkled twice daily and once on Sunday. The teams were hired at $110 per month, and water was supplied by the local company at 1c. per 100 gal. of water, each tank being equipped with a meter.

The sprinkling season covered six months. Under the charter the cost of sprinkling each street has to be kept separately, so that special report blanks were necessary, entailing a large amount of bookkeeping. Daily reports showed the amount of water used on the street and the time spent. Monthly reports were made up from these daily records, showing for each street the total amount of water and the labor cost for hauling and sprinkling.

The cost estimate for the season was based on 100 sq.ft. of surface, including street intersections. The entire cost, including intersections, was assessed against the abutting property on the frontage basis. Corner property was assessed in full for each frontage. The average cost to the property owners was 0.2 to 0.57c. per front foot on the paved streets and 0.2 to 0.51c. on the unpaved streets. The cost under city force account in 1915 was $2,985. In 1914, under the contract system, the cost was $3,086. This latter was for less frontage and yardage sprinkled, while the city furnished the wagons and paid half the water cost.

Simple Type of Concrete Bridge for Grade Crossings

By C. W. Martin*

A type of concrete bridge well adapted to grade-crossing work, particularly where the crossing is over a cut, has been used in two bridges at St. Louis, Mo., over the tracks of the Missouri Pacific Ry., one at Arsenal St. and the other at Columbia Ave. The accompanying view illustrates the latter structure, and the main description and the drawings herewith relate to the former.

The railway tracks are situated in an excavation varying in depth from 21 ft. at Arsenal St. to 17 ft. at Columbia Ave., so that at the former a vertical clearance of 18 ft. over the tracks is obtained with a street grade at about the general ground level, while at Columbia Ave. a street embankment of about 5 ft. is necessary to obtain a like clearance. At Arsenal St. the track alignment is practically normal to the street center line, but on account of about 18° of curvature intervening the track alignment at Columbia Ave. makes an angle thereto quite oblique. In general the tracks are 14 ft. centers, and the excavation side slopes are, say, 1¾ to 1. Therefore the three spans of 31 ft. 6 in. length each, at Arsenal St. and two end spans of 32 ft. and one center span of 33 ft. 6 in. length at Columbia Ave. provide normal side clearances of 8 ft. from the columns to the track center lines and allow the construction of the abutments well back and near the top of the excavation slopes. Both bridges extend the full width of the street, 60 ft., and their overall lengths from end to end of abutment wing walls are 110 and 140 ft. for Arsenal St. and Columbia Ave. respectively.

Design of Abutments, Piers and Deck

All abutment and wing walls are of gravity section and save the back walls, which abut the ends of the deck slabs and retain that portion of the roadway embankments lying back and above the bridge seats, are without reinforcement.

Each pier bent consists of a series of columns supporting a transverse beam on which the deck is carried. Footings for all columns extend to rock. The beams overhang the outside columns of each bent so as to extend longitudinally just beyond the outside stringers of the decks. The longitudinal reinforcement of the beams is so arranged as to provide resistance to negative as well as to positive bending moments, and at points of maximum shear the bars are bent diagonally and anchored with hooked ends. This reinforcement is augmented throughout by a system of vertical stirrups. At the ground level each pier bent is protected by a guard block that collars the columns and parallels the railway tracks. Throughout, the arrangement of columns, footings, and guards is such as may permit a future reduction of 4 ft. in the grade of the railway tracks.

The deck structure consists of three unit spans of integral slab and stringer construction. For each span the

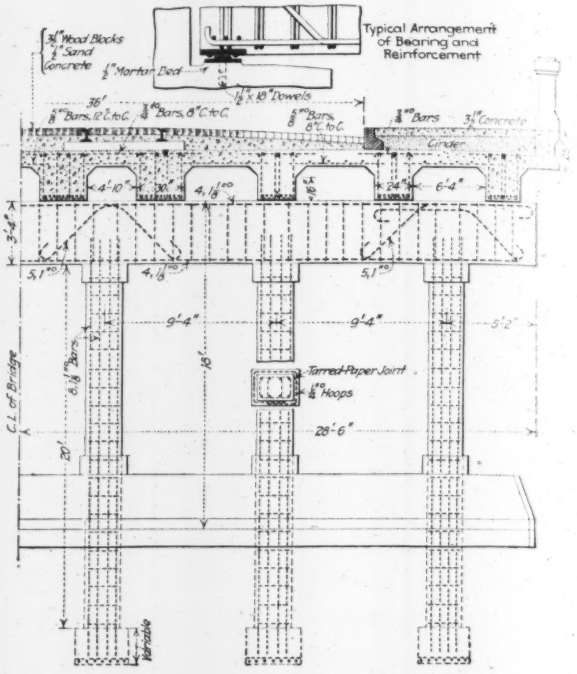


FIG. 2. DETAILS OF ARSENAL ST. BRIDGE AT ST. LOUIS

FIG. 1. COLUMBIA AVE. BRIDGE ACROSS THE MISSOURI PACIFIC RY. TRACKS AT ST. LOUIS

*Street Department, City of St. Louis, Mo.

slabs are reinforced transversely for both negative and positive moments, while the stringers are reinforced longitudinally for positive moments only. At the abutments all stringers are furnished with cast-steel shoes and bedplates, and provisions are made for temperature distortions, as shown in the detail in Fig. 2.

The principal structural elements of each of the three 31-ft. 6-in. deck-span units at Arsenal St. consist of 10 lines of stringers and a 10-in. slab. The top line of a transverse section of the slab is straight at a depth of 12 in. below the street grade. This 12-in. clearance allows track construction with 7-in. rails and 4-in. special steel crossties. It also necessitates the use of considerable filler concrete over the slab for the bases of pavements and walks. The stringers are 24 and 30 in. wide, 22 in. deep below the bottom of the 10-in. slab and, save the four under the tracks, are at 6 ft. 4 in. centers.

The longitudinal reinforcement of the stringers consists of rods placed in one layer with approximately one-half the number of these rods bent up near the stringer ends and anchored within the compression area of the concrete section. Throughout, the stringers are further reinforced by vertical stirrups which circumscribe the longitudinal bars and, by hooked ends, engage the top system of transverse rods in the slab.

In the design of these bridges the anticipated settlement of the soil-bearing abutments was a cardinal factor for the adoption of a type of construction which provided "clean-cut" full-width construction joints in the deck structure over the center line of each pier beam—a type in which longitudinally, simple beam action might truly prevail.

Both bridges were designed for the following surface loadings: (a) An 18-ton road roller on roadway stringers; (b) a 16-ton concentrated wheel load on roadway slabs; (c) a uniform roadway load of 125 lb. per sq.ft.; (d) a uniform sidewalk load of 100 lb. per sq.ft.; and (e) for the Arsenal St. bridge only, a 50-ton street-railway cinder car on track stringers.

Critical combinations of these loads augmented by an arbitrary impact allowance of 30%, together with the dead weights, generate figured maximum unit stresses in various elements of the structures not exceeding the following pounds per square inch: (a) Axial compression in concrete, 450; (b) flexural compression in concrete, 650; (c) shear in concrete, 120; (d) tension in steel, 16,000; (e) bond in plain bars, 50; (f) bond in deformed bars, 120.

Quantities and Prices

For purposes of comparison the following details of quantities and prices are given:

	Arsenal St.	Columbia Ave.	
Span lengths	31 ft. 6 in.	32 ft. 0 in.	33 ft. 6 in.
Overall depth	3 ft. 8 in.	3 ft. 2 in.	3 ft. 2 in.
Quantities per sq.ft.:			
Concrete, structural, cu.ft.	2.00	1.65	1.65
Reinforcement in slabs, lb.	4.10	3.80	3.80
Reinforcement in longitudinal stringers, lb.	7.50	5.40	5.70
Reinforcement in stirrups, lb.	2.55	1.80	2.00

The average dead-weight of deck structure complete with pavements, etc., is 270 lb. per sq.ft. at Columbia Ave. and 400 lb. at Arsenal St. This increased weight at Arsenal St. resulted largely from the filling required under the pavements and considered together with the street-car loading makes manifest the necessity of increased structural quantities over those required for the somewhat longer deck units at Columbia Ave.

	Quantities		Unit
	Arsenal St.	Columbia Ave.	Price
Earth excavation	6,300 cu.ft.	9,900 cu.ft.	$0.06
Rock excavation	110 cu.ft.	20 cu.ft.	.30
Concrete, Class A	6,160 cu.ft.	11,070 cu.ft.	.33
Concrete, Class B	11,560 cu.ft.	10,250 cu.ft.	.35
Concrete, Class C	360 cu.ft.	500 cu.ft.	.90
Steel reinforcement	103,300 lb.	85,000 lb.	.025
Steel castings	3,860 lb.	2,180 lb.	.07

This makes a cost of $9,700 for work done under contract at Arsenal St., and $10,600 at Columbia Ave. To these figures must be added items of $1,800 for costs of labor and material expended by the city in removing the old bridge and its abutments and the construction of pavements, curbs and walks at Arsenal St., and $1,100 at Columbia Ave. for the construction of embankments and roadway pavement. These items, together with others for plans, inspection, etc., make the total cost of each of the two finished bridges approximately $12,400.

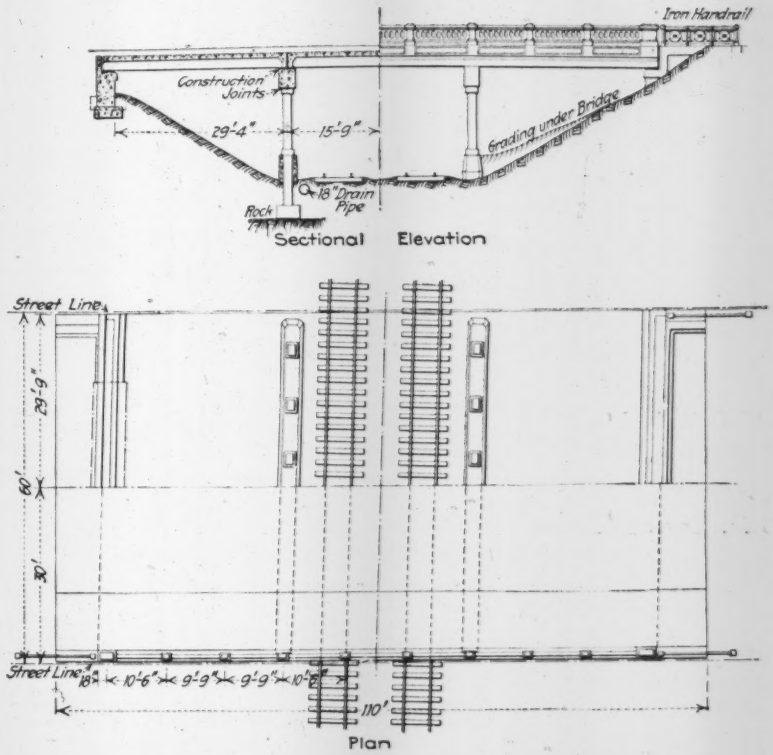


FIG. 3. OUTLINE OF ARSENAL ST. BRIDGE AT ST. LOUIS

Formwork Notably Simple

Primarily on account of the limited clearance over the steam-railway tracks, 10-in. 25-lb. steel I-beams were used for the principal longitudinal members of the falsework spans. At the bridge ends they rested directly upon the abutments and at the pier bents were supported by 6x12-in. timbers capping 6x8-in. posts which acted also as studding for the side sheathing of the pier-beam forms. At midspan points additional support was provided by

framed bents. Throughout, the I-beams were placed in the bays between the concrete stringers and straddled by inverted 5/8-in. U-bolts that carried a transverse system of underhung 3x6-in. joists on which the form sheathing was framed. This arrangement of forms provided a vertical clearance over the railway tracks of 17 ft. 3 in., which is only 9 in. less than that of the finished bridges.

General Conclusions to Design

These bridges are of fairly good appearance and in general of satisfactory detail. The columns and the arrangement of end bearings for stringers at the abutments are two structural features offering marked chance for betterment in future bridges of this type. While the sectional dimensions of the columns are such as to provide adequate axial strength, improved architectural appearance as well as augmented resistance to sidewise impact would have resulted from their enlargement. Though frequent inspection of stringer ends has thus far revealed no evidence of inadequacy in the used arrangement of end shoes and bearing plates, it now appears to the writer that somewhat improved detail might have been obtained by placing the shoes farther forward along the bottom of the stringers so as to secure more positive anchorage for them. Or probably still better, these cast-steel bearings might have been omitted and the stringers anchored directly into some type of flexible abutment.

Both bridges were built under the direction of the Bridge Division of the Street Department of the city, under C. M. Talbert, Street Commissioner; L. R. Bowen is Bridge Engineer. The writer designed the structures and was in charge of their construction for the city.

Rail Joints on Philadelphia & Reading Railway

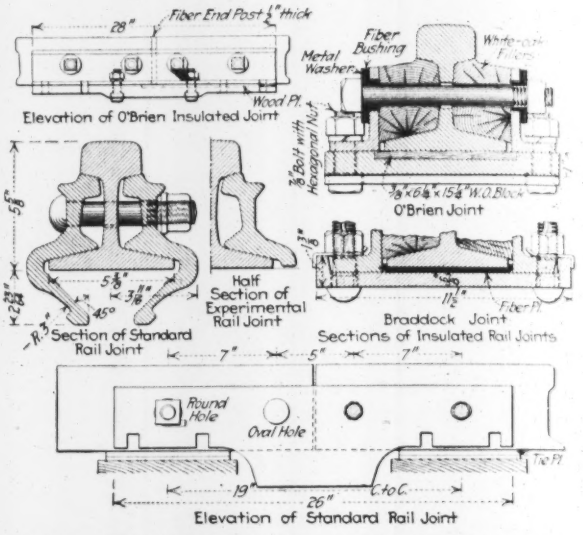
The standard splice adopted by the Philadelphia & Reading Ry. in 1915 for its 100-lb. rails consists of short splice bars of a heavily ribbed section, with four bolts, as shown in the accompanying drawing. The bars are 26 in. long and are of the "100%" pattern, in which the middle part has an inclined flange extending inward beneath the rail, as shown. This construction is for the purpose of increasing the stiffness of the joint at the middle. The splice bars are slotted for the track spikes and have the bolt holes spaced 5 in. for the inner and 7 in. for the outer bolts.

The bolts are 1 in. in diameter, with oval necks, and the heads are shaped to fit the ribs of the splice bars. Spring nutlocks and heavy square nuts are used, the nuts being 1 in. thick for ordinary bolts and 1 1/8 in. for those of heat-treated steel. The bolts are placed with heads alternately on the inner and outer sides of the joint, as is now very general practice. The bolt holes in the splice bars are alternately 1 1/16 in. in diameter and 1 1/16 x 1 5/16 in. oval, while the holes in the rails are 1 3/16 in. in diameter.

The railway is experimenting also with a joint that has no part extending beneath the rail, as shown in the drawing. This arrangement will permit renewal of rails without any disturbance of ballast or adjustment of the positions of the ties to fit the joints. Except for the section of the splice bars this joint is similar to the standard joint just described.

For insulated rail joints two standard designs were adopted in 1915—the O'Brien and the Braddock joints. The former, which is the more generally used, is shown on the drawing. It consists of a heavy base plate, 2 x 10 1/2 in., recessed at one end to receive a 3/8-in. white-oak bearing plate for one rail. Angle splices are bolted to the sides of the base plate, and between them and the webs of the rails are white-oak fillers. Slotted holes in the horizontal legs of the angles permit lateral movement in tightening up the splice bolts.

The long splice bolts are fitted with fiber bushings where they pass through the splice angles. The joints have four or six bolts (all with 28-in. splice bars), the spacing being 5 1/4 and 7 in. for the former and 4 1/4, 4 and 5 in. for the latter. The rail ends are separated



RAIL JOINTS ON THE PHILADELPHIA & READING RY.

by a 1/2-in. fiber end-post shaped to conform to the rail section.

The Braddock joint is very similar in design, but has a base plate of channel section, with ribs outside the splice angles. Each angle is held by a single vertical bolt. The base has no pocket for a bearing plate, but a steel shim is placed beneath one rail and a fiber shim beneath the other, each of these being 3/16 in. thick.

For drawings of these joints we are indebted to Samuel T. Wagner, Chief Engineer, Philadelphia & Reading Ry.

A Great Irrigation System in India was opened officially in December. It comprises three separate but connected canals—the Upper Jhelum, the Upper Chenab and the Lower Bari Doab. These aggregate 322 mi. in length, with about 22,645 mi. of auxiliary channels and laterals. They provide for the irrigation of about 2,200,000 acres of arid land in the Punjab province, in the northern portion of the country. As described in a paper presented before the Institution of Civil Engineers by Sir John Benton, the eastern portion of the Punjab had a tract of 1,500,000 acres of arid but good land which could not be irrigated from any water-supply near at hand, owing to previous utilization and reservation of such supply. On the western side of the province the Jhelum River provided a large and unused amount of water. To deliver it to the arid lands necessitated extensive and difficult works, with the crossing of two large rivers and numerous mountain streams and torrents. Bridges are spaced at average intervals of 1.6, 1.5 and 3.4 mi. for the three canals respectively. The regulating works, etc., are largely of brick. Chambers in the floors provide settling basins for the silt and so reduce erosion of the masonry floor. Inspectors' houses are placed at intervals of about 10 mi. There is a complete telegraph system. Flour mills have been located at some of the falls on the canals. The project has been carried out by the provincial government of the Punjab at a cost of about $35,000,000.

Four Years of Concrete Roads in One County

The differences in construction and service results of the concrete roads in Lake County, Ohio, were detailed in an excellent paper read at the annual meeting of the Ohio Engineering Society on Feb. 10 by H. P. Cumings, County Surveyor of Lake County. Extracts from the paper follow:

The first concrete road in Lake County was built in 1912 under state specifications and state supervision, on the road leading south from the village of Madison; its length is 1.41 mi.; width, 12 ft.; thickness of concrete, 6 in.; cross-slope, 2 in. The soil is a heavy clay. It was a one-course job, and the mix specified was 1 : 1½ : 3½. The surface was finished with a wood float; and after the entire length of pavement was completed, it was cleaned as thoroughly as possible and given a surface treatment of tar.

The cleaning process was no joke, as the pavement had been covered with clay to aid in curing, and a considerable part of it had been traveled over for several weeks. Steel brooms, reed brooms and barn brooms were used, and we hoped that what clay didn't come off would be absorbed. Perhaps this was the case, for over the greater part of this pavement the tar stuck well.

Over a stretch of several hundred feet in one location it peeled off badly the first winter, without any reason that I was able to discover. This section was repaired the following spring and has since stood as well as the other part. A half-dozen cracks have appeared. This pavement has convinced me that concrete can be laid on a clay soil and not break up from the effects of freezing and thawing.

The contract price of this pavement was $11,970. The contractor assures me, however, that those figures do not indicate what the pavement actually cost.

Dead Rock Caused Holes in Second Pavement

The second concrete pavement was laid in Lake County in the summer of 1913. It is 2.21 mi. in length, 12 ft. in width and 6 in. thick. It was built under state specifications and is a two-course job, the cost being about $25,000. The soil is clay. The foundation course consists of a 1 : 2½ : 5 concrete 4 in. deep; the top course is of a 1 : 1½ : 3 concrete 2 in. in depth.

Local creek gravel was used as the coarse aggregate for the base; the fine aggregate was a creek sand from the same location as the gravel and showed up exceptionally well in tests. The coarse aggregate for the top was obtained by crushing boulders collected from the near-by fields. A large percentage of these boulders was granite and supplied a crushed rock of very good quality. There were, however, among them some of a soft, or dead, quality. This accounts for the fact that numerous holes have appeared in the surface of certain parts of this pavement.

Before the concrete was opened to traffic, the surface was thoroughly broomed with steel brushes, and a coating of tar and granite screenings applied. When the supply of screenings from the crusher plant was exhausted, the remainder of the surface was covered with fine silica gravel. The screenings seemed to give the more satisfactory surface. The tar coating, however, began peeling off badly the first season after being completed, and today about one-half the surface is bare. This pavement has had a considerable amount of heavy traffic.

Three blocks have broken badly, owing to settlement of a fill. Considerable cracking has occurred, about 100 blocks being broken out of a total of 390. Some displacement has taken place at joints, due, I think, chiefly to settlement of blocks.

In the summer of 1914 an extension of this pavement was built. This extension was 1.34 mi. in length and was similar in every way to the one just described, except that the top course was a 1 : 1 : 2 concrete; slag was used for the coarse aggregate in the base, silica gravel in the top for about one-half the job and hard limestone in the top for the remainder. No tar was used.

The surface is today in almost perfect condition, but a considerable amount of cracking has taken place, about 100 blocks being broken. A very close examination is required to detect the difference in the surface where the silica gravel was used and where the limestone was used. The contract price of this work was $17,115. In both the two last described pavements the cracks are about equally divided between longitudinal cracks and transverse cracks, but in each case nearly all occur in the middle one-third of the block.

Experience with Pebble Wearing Surface and Poor Sand

Also in the summer of 1914, in the township of Madison, 9.8 mi. of county road were built under two contracts—one for the South Ridge Road, the other for the Lake River Road. The South Ridge road pavement is 16 ft. wide, 6 in. thick at the edges and 7 in. in the center, the surface having a crown of 1½ in.; it has one course—1 : 2 : 3½.

The fine aggregate was a dredged lake sand. Our only difficulty in keeping this sand up to requirements was to get sand that was sufficiently coarse, and it was necessary to reject a number of carloads on this score. The coarse aggregate was the very hard, round, smooth pebble that we know as silica gravel. The condition of this pavement at the present time would indicate that a concrete built with this pebble offers a wearing surface almost indestructible under the abrasive wear of ordinary traffic, but also gives a body lacking in tensile strength.

Reinforcement Prevents Cracking

Some 90 blocks were reinforced with American Steel and Wire Co.'s triangle mesh No. 28. These blocks were in wet or springy places, on fills or where sewer trenches had been excavated. Out of these 90 blocks (33⅓ ft. each) about a half-dozen have fine cracks. These have not opened up so as to cause any damage whatever. You can follow a longitudinal crack down the center of this pavement block after block, till it comes to a block that was reinforced because of a sewer trench. This block will be perfectly sound, and the crack will begin again on the other side. A few transverse cracks have appeared, but the most of the cracks are longitudinal. These cracks nearly all appear in the middle one-third of the block.

The first half-mile of this pavement was laid with ⅜-in. expansion strips at the joints, with no armor plates. The edges of the blocks at these joints have rounded off, but the raveling has increased very little since the first few months of traffic. After the first half-mile was laid, all joints were armored with Kahn plates. These joints

are in practically perfect condition, except where the concrete was left too high and floated over the plates, in which cases the edges of the blocks have rounded down to the plates.

The cost of this road was $97,909.37, divided into four general groups as follows: Grading, $11,465.50; drainage, $13,082.17; pavement, $71,738.26; masonry and culvert repairs, $1,623.44.

On the Lake River road the same difficulties in securing a sufficiently coarse sand were experienced in the early shipments for this road as on the South Ridge road. Some of this defective sand got into the first mile. As soon as the last block laid was three weeks old, hauling over it of material for the 2½ mi. north of it was begun. At numerous places in this mile of pavement the surface has worn down under traffic, showing depressions at these spots of from ½ to 1 in. Most of this wear seems to have occurred during the first few months after the pavement was opened to traffic, very little additional wear having occurred at these places during the past summer.

We believe some of these defects are due to poor sand, but a greater cause is the breaking of the bond in the surface of the concrete by too heavy traffic before the concrete was old enough to withstand this weight. I have observed that a traction engine with lugs on the wheels, which would chip the pavement in passing over it the first season after it was opened to traffic, will now pass over it without leaving a mark, showing conclusively that the pavement is not at its best for a long time after it is laid and that traffic should be kept off it as long as a long-suffering public can be made to suffer.

The cost of this road was $72,841.07, divided into the four groups as follows: Grading, $5,992.04; drainage, $10,344.45; pavement, $56,430.58; masonry, $64.

New Pavement Reinforced; Crushed-Rock Aggregate Used

Our experience with the Madison pavements, not only in the matter of the cracking of the concrete, but also because of difficulties we experienced in securing a uniform grade of gravel for a coarse aggregate, determined us to use only a crushed rock for the coarse aggregate in the next concrete pavement to be built in Lake County, and also determined us to reinforce all future concrete pavements. Following out this decision, contracts were let in the spring of 1915 for paving with concrete three roads in the township of Perry, aggregating 9.73 mi.

The concrete pavement is 16 ft. wide, 6 in. thick at the edges and 7½ in. in the center. The concrete was laid in two courses, the base being 4 in. at the edges and 5 in. at the center; the top course is 2 in. at the edges and 2½ in. at the center.

These pavements are now going through their first winter. It is too early to be able to say what they will eventually show. They have already passed through four periods of moderate freezing—one period reaching zero—followed each time by a sudden thaw and complete break-up. Up to the present time I have not been able to discover one crack or defect of any kind in the entire 7.5 mi., though I have made several careful inspection trips over them; nor have any been reported.

Out of six roads herein described, each somewhat different in its construction from the other, we have pinned our faith to the Perry construction.

Diagram Giving Excess Loss of Head in 90° Bends

By Frank S. Bailey*

Losses of head owing to bends in pipe are of so much importance in complicated systems of piping, such as fire-sprinkling systems, sewage-sprinkling filter systems, piping in pumping stations, automatic dosing tanks and the like, that it is hoped and believed the accompanying diagram will prove of much assistance to engineers requiring such information. This hope and belief are increased by the fact that the writer has not seen any other diagram or table in which excess loss of head due to bends on account of varying velocities of water in pipes can be found so rapidly as by means of this diagram.

The diagram was prepared by the writer for use in the office of Metcalf & Eddy, and is published with their permission. It is based on the formula proposed by W. E. Fuller, of Hazen, Whipple & Fuller, in a paper by him which appeared in the *Journal* of the New England Water-Works Association, for December, 1913.

Mr. Fuller says, in effect, that the experimental data are insufficient to settle some of the questions involved in the loss of head in bends, and that further experiments may show the desirability of modifying the formula. This formula, however, agrees fairly well with most of the experiments made recently, and in the opinion of the writer is without doubt the best one which has thus far been suggested. In deriving it, the experiments of Williams, Hubbell and Fenkell, at Detroit, of Schoder, at Cornell, Davis, at the University of Wisconsin, Brightmore, in England, and others were considered.

The formula is $H_B = KV^{2.25}$, in which

H_B = Loss of head in feet in a 90° bend in excess of the loss in a straight pipe of length equal to the length of the curve at the center of the pipe;

K = A coefficient, different for bends of different radii;

V = Velocity of water in ft. per sec.

In the diagram here presented the value of H_B is not given, but instead the loss of head in feet due to a 90° bend in excess of the loss in straight pipe equal in length to the tangents of the curve at the center is shown. In practice it is better to know this loss, as, to quote Mr. Fuller, "It is most convenient to compute the loss of head in pipe lines by taking the actual lengths of the tangents as straight pipe, finding the frictional resistance in it and then adding the excess resistance due to curves and other specials." The computations for the excess loss shown on the diagram were made as follows:

The loss of head caused by the bend, in excess of friction head for a length of straight pipe equal in length to the curve, was computed from formula $H_B = KV^{2.25}$, and the friction in a length of straight pipe equal to the difference between the length of the tangents and length of curve was then deducted. Frictional resistance in straight pipe was computed by the Hazen-Williams formula, with $c = 100$, this value of c representing average pipe after being in use a few years.

The diagram shows the excess loss as described for 90° bends of from 1- to 48-in. inside diameter and for velocities of from 2 to 8 ft. per sec. for 1- to 4-in. sizes, and also

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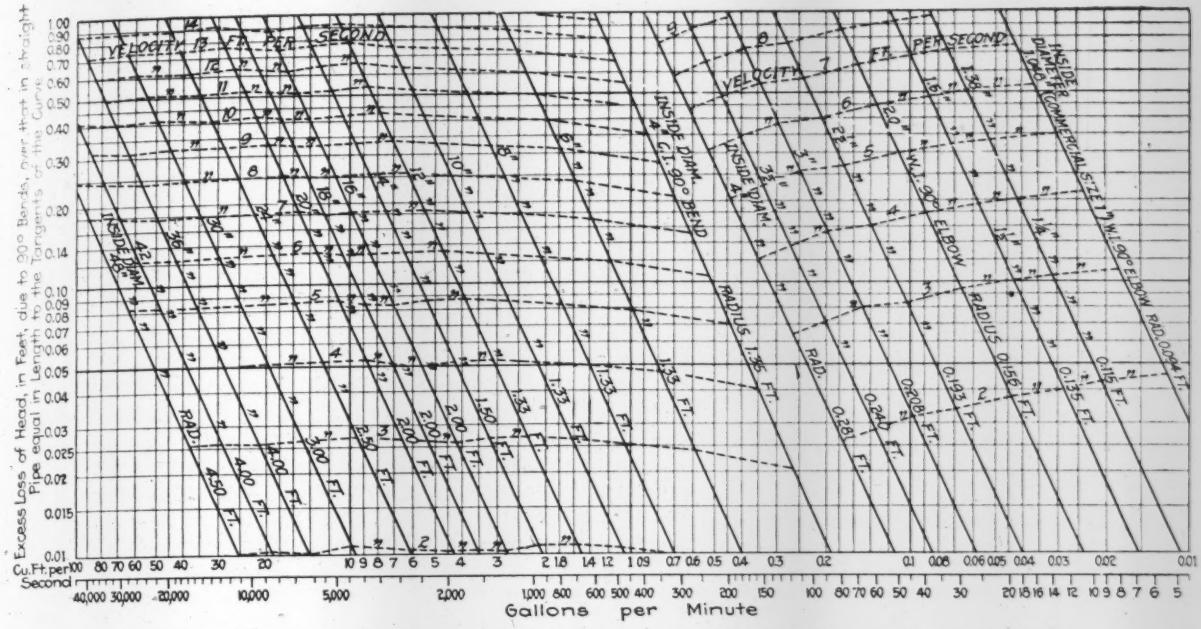

DIAGRAM GIVING EXCESS LOSS OF HEAD IN 90° BENDS IN CAST-IRON WATER PIPE
Based on formula proposed by W. E. Fuller

from 2 to 14 ft. per sec. for 4- to 48-in. sizes; it also shows the loss in cubic feet per second and in gallons per minute when different quantities are flowing through the bend.

The radii used in the computations for the various sizes are given on the diagram. For the 1- to 4-in. wrought-iron sizes they are approximately the same as found in ordinary "malleable fittings"; for the 4- to 36-in. cast-iron bends they are American Water-Works Association and New England Water-Works Association standards; for 42- and 48-in. bends they are those given in the catalogs of the United States Cast-Iron Pipe and Foundry Co.

In computing the losses in the bends of 4- to 1-in. diameter, it was assumed that the inside diameter of the bend was the same as the inside diameter of the pipe. In some small bends the diameter of the bend is larger than that of the straight pipe, and in such cases the loss of head due to the bend will be greater than if the bend and the straight pipe were of the same diameter.

In discussing E. W. Schoder's paper, "Curve Resistance in Water Pipes" (*Trans.* Am. Soc. C. E., Vol. LXII, p. 112), G. J. Davis, Jr., gives results of experiments with a bend of $2\frac{1}{16}$-in. inside diameter attached to straight pipe of the same size, and with a bend of $2\frac{3}{8}$-in. inside diameter attached to straight pipe of $2\frac{1}{16}$-in. diameter. Both bends had a radius of $1\frac{1}{2}$ in. to the center line. He found the loss of head in the bend of $2\frac{3}{8}$-in. diameter to be about 1.8 times the loss in the bend of $2\frac{1}{16}$-in. diameter. This is worth noting, as it might be supposed that the bend of larger diameter would not cause any greater loss of head.

The reading of the diagram is simple. If, for instance, it is desired to know the excess loss of head due to a 90° standard-pattern bend in a 12-in. pipe with 5 cu.ft. per sec. passing through it, all that is needed is to find the intersection of the vertical 5 c.f.p.s. line with the sloping line marked 12-in. inside diameter and then follow parallel to the horizontal lines to the excess loss of head scale, where the excess loss is found to be 0.15 ft. The dotted lines representing velocity show that in this case the velocity in the pipe is about 6.4 ft. per sec. It is interesting to note that the velocity curves are nearly horizontal and that the same velocities in the different sizes of pipes cause similar excess losses of head, the radii of course varying as shown.

It may be of value to note that for bends of 45° Mr. Fuller suggests using three-fourths the loss of head in 90° bends of the same radius; and for $22\frac{1}{2}$° bends, one-half that in 90° bends of the same radius.

Sidewalks in Steep Streets

In hilly towns the convenient arrangement of sidewalks on steep-grade streets forms a special problem. This problem was discussed at the recent annual meeting of the Minnesota Surveyors and Engineers Society by M. E. Chamberlain, of Montevideo, Minn. It is not always realized that on sidehill streets the sidewalk need not follow the street grade or be at a uniform height above the roadway. In some cases very good results have been obtained by varying the transverse slopes and making adjustments in the sidewalks, curbs and gutters. A slope of 20% from the curb to the walk is not objectionable.

For the benefit of property on the low side of the street the curb may be located nearer to the low than to the high side, thus permitting a reduction in street-grade elevation. In extreme cases the walk has been narrowed or even placed at the curb. These variations in design minimize the damage to elevated or depressed residence property. On sidehill streets, also, another problem is often the unnecessary width between curbs. A width of 24 ft. is considered ample for residence streets.

Sidewalk steps are often necessary on steep-grade streets in hilly towns. As people walk faster out of doors, the steps should be considerably wider than those in buildings; and the author advocated 15-in. treads and 6-in. risers. In one Minnesota town there is a long flight of steps with 40-in. treads. Steps with 15-in. treads, in series separated by landings, would have been better.

Lowering Broadway Tunnel in Los Angeles

By R. W. Stewart*

The most direct route from the business district of Los Angeles to large residential sections and populous suburbs lying north is through the Broadway tunnel. This tunnel was built in 1900. It is 760 ft. long. The regrading of Broadway, although it involved a cut of 38 ft. maximum depth, left a 7.5% grade ascending north to the south portal of the tunnel. From the summit at the south portal the tunnel descended northerly on a 6% grade with an unpaved roadway.

The dense automobile traffic of recent years showed a tendency to detour around the hill through congested streets, indicating necessity for improvement of the tunnel route to make it serve the purpose for which it was intended. The matter was taken in hand by the City Council, and it was decided to lower the tunnel 20 ft. at its south end and 3 ft. at its north end, thus reducing the grade in the tunnel to 3.8% and in the block south to 2.9%. Since the old tunnel had been dark and the walls were discolored by water seeping through the masonry, it was decided that, in addition to the grade change, the interior should be lined with white-enameled tile and illumination provided.

Lining with tile required that all irregularities in the interior surface be corrected. The inwardly curving footings and slight irregularities in the old tunnel would have made it a tedious and expensive undertaking to true up the masonry; and the tunnel, if thus treated, would have had a freakish appearance, one end being much higher than the other. Because of these conditions and because of the large economy in area of tile lining and the reduction in lighting expense, it was found preferable to construct a new tunnel arch under the old one rather than undertake to treat the old surface. The details of the design are well illustrated by the figures and need little comment.

The new arch has a uniform thickness of two rings of brick, and being loaded only with its own weight, it was given a compound curvature such that its neutral axis deviates but a small fraction of an inch from a common catenary.

Provision for drainage was made by placing 3 in. of broken stone behind the underpinning walls. The most convenient and economical way of doing this work was found to be by fastening 3-in. vertical wooden strips to the face of the excavation, using thin metal clips and long nails driven into the hard clay. Thin sheet iron was then placed against the wooden strips, and the crushed stone was placed behind it in advance of the concrete, the sheet iron being left in the work. The bottom of the stone backing was drained at frequent intervals by vitrified pipes, as illustrated. Iron weep pipes were also placed, as shown, to dispose of any water that might seep through the old tunnel.

The underpinning walls were constructed in sections 10 ft. in length, the specifications providing that excavation could not be commenced for any section when there was a point within 70 ft. of such section where the tunnel was not supported either by undisturbed earth or by concrete that had set 10 days.

The new brick arch was built on traveling centering. Two sets of forms were used—one 18 ft. long and one 20 ft. long. Bricklaying was commenced both at the south end of the tunnel and at the center. One crew of 5 bricklayers and 12 helpers completed one 18- or 20-ft. section each day, working on alternate days in the north and south portions of the tunnel, so that each section of

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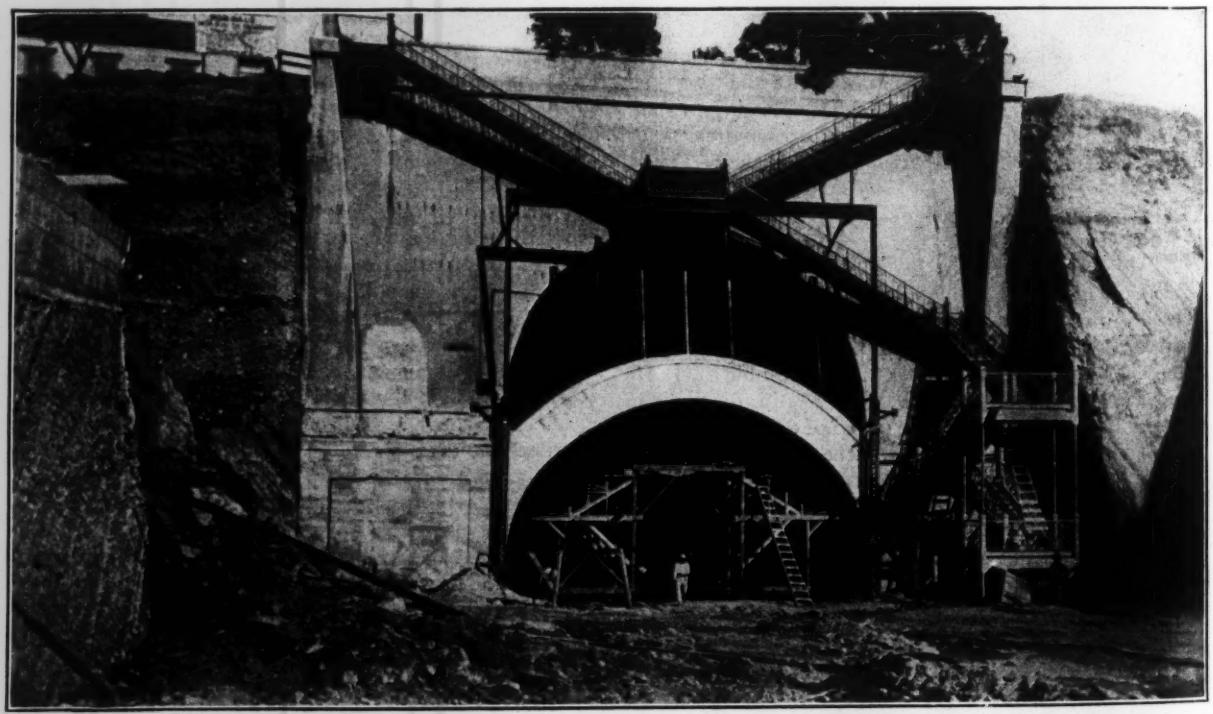


FIG. 1. PARTIALLY CONSTRUCTED SOUTH PORTAL OF TUNNEL NOW COMPLETED UNDER AN OLD TUNNEL

brickwork had one day to set before the centers were struck. Some concern had been expressed about the thinness of the arch and the length of the span; but this anxiety proved unfounded, as no trouble whatever was experienced. The top of each section was plastered the evening of the day it was laid. After all brickwork and top plastering were finished, the entire upper surface was coated with hot asphalt. In doing this, barrels of asphalt were rolled around on the arch without vibration being noticed.

Tile-setting likewise was carried on in two sections, using two 50-ft. movable scaffolds. Great care had been exercised in constructing and setting up the centering for the brickwork, in order to obtain a perfectly true bore throughout the tunnel. As a result, it was possible to start the tile at the bottom on both sides of the tunnel and make a perfect closure at the top without the use of variable-width key tile, and this was done. The procedure in underpinning and some of the costs were as follows:

1. A steam-shovel cut about 7 ft. deep was taken out through the entire length of the tunnel, leaving enough material along the walls to support them. This first excavation amounted to 6,554 cu.yd., which was taken out in 33 days by a crew consisting of 1 shovel engineer, 1 foreman and 2 laborers. The total labor cost to load the material on wagons and motor trucks was 7¼c. per cu. yd. The earth was hard, stratified clay or soft shale, which would stand vertically with excellent supporting qualities.

2. Underpinning sections were excavated at points 10 ft. apart, and the underpinning concrete was poured section by section. The concrete mixer was mounted on wheels, so that it could be pulled by horses from section to section. Its crew consisted of 1 foreman, 1 mixer engineer and 5 or 6 laborers. The average day's work was 39.3 cu.yd., the labor cost of mixing and placing being 61.5c. per yd. This cost refers only to the concrete that was chuted from the mixer directly into the walls. Concrete for the top layer, which was about 1 ft. thick, was mixed by hand to a stiff consistency and rammed in horizontally, to insure firm contact with the bottom of the old masonry.

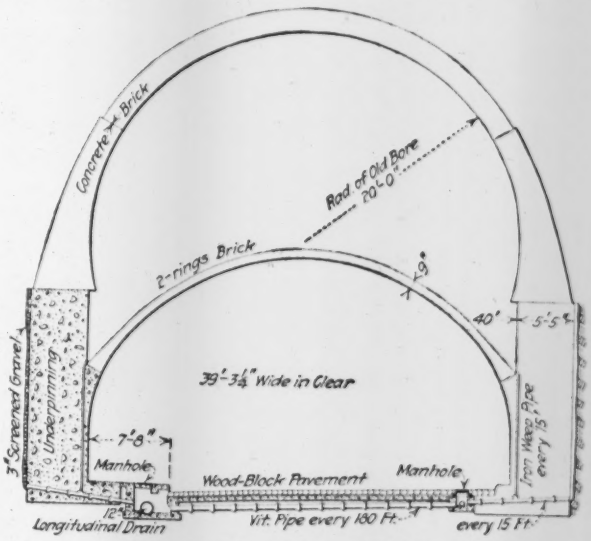


FIG. 3. CROSS-SECTION OF OLD AND NEW TUNNELS, SHOWING DRAINAGE SYSTEM

3. After completion of the underpinning, another steam-shovel cut removed all earth between the walls down to pavement subgrade.

4. Stone cutters working with hand tools did such cutting away of the old concrete as was necessary to make it clear the new brick arch. This work was necessary in the northerly portion of the tunnel, where the grade change was insufficient to cause the new arch to clear the old, particularly at the abutments.

5. After trimming the old masonry, the centering was erected and the new arch laid. The average number of bricks laid by the daylight shift was 2,230 per mason per day. The single mason with 3 helpers, who worked on the night shift, laid 900 bricks to key the arch and plastered 18 or 20 lin.ft. of extrados each night. The labor cost of wetting, hoisting and laying the brick (at $5 per day of 8 hr. for bricklayers and $2.50 per day for laborers) was $6.86 per 1,000. This was accomplished under specifications that were very strict as to quality of work. All brick were laid as stretchers with broken joints, and when any unfilled joints were found, both the defective work and the mason were removed.

The plans were prepared in the city engineer's office under the direction of Homer Hamlin, City Engineer. The contractor for the entire work is Lynn S. Atkinson, of Los Angeles. The total contract price is $189,900.

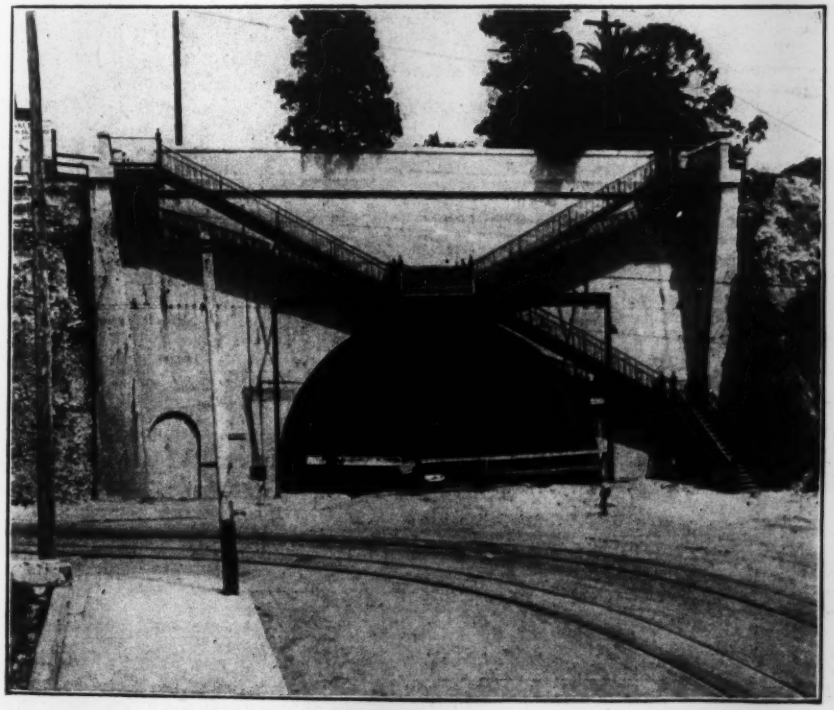


FIG. 2. SOUTH PORTAL OF OLD TUNNEL BEFORE LOWERING

New Coal Shipping Station at East St. Louis, Illinois

A coal shipping plant of novel design, for the purpose of transferring coal from railway cars to river barges, is being built at East St. Louis, Ill. It will have a capacity of shipping 150 tons per hr. In design it is based upon the principle of the transfer inclines for car-ferries on the Ohio and Mississippi rivers.

From the coal yard a supply track extends along an embankment adjacent to the inner harbor line, and having a trestle over the head of an incline which extends (at right angles to the supply track) down the river bank. The incline has a grade of 12%, and on it is a track of 6 ft. gage upon which rides a cradle built of structural steel. The deck of the cradle does not come to the level of the deck of the boat, as for a car-ferry, but is on an ascending grade of 20.35% in order to have the end at proper height for dumping. The cradle is carried on eight 33-in. car wheels and weighs about 22 tons. It is moved up and down the incline to follow the stage of water level in the river.

The railway car on the trestle of the supply track dumps the coal into a track hopper of 12 tons capacity from which it is discharged as required into a transfer car. This car runs on the 6-ft. track of the incline and up the cradle to a dumping platform, whence it discharges its load into the barge below. The general arrangement of the plant is shown in the accompanying drawing. The tracks are laid with 80-lb. rails. On the cradle incline the ties are 8x8 in., 10 ft. long, 18 in. c. to c., with spacing blocks 6x8 in. to prevent creeping.

The operation is effected by means of three cables, two to handle the coal transfer car and the other to handle the cradle. The drums for the cables are arranged in the operating house and are driven by gearing from an electric 35-hp. motor. One man controls the movements of the drums by the levers, and also operates the gate

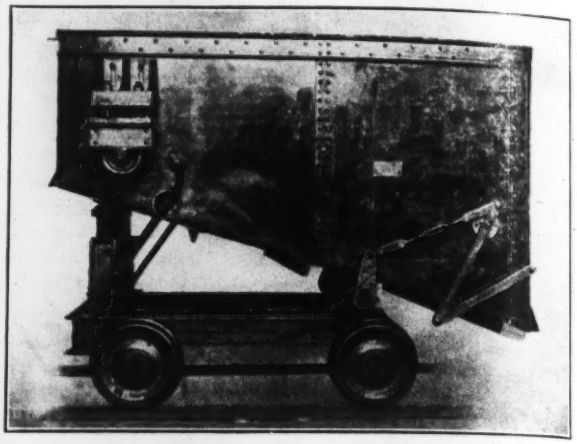


FIG. 2. 12-TON DUMP CAR FOR COAL-SHIPPING CRADLE

of the track hopper by means of a hand wheel with sprocket wheel and chain.

The hauling cable *A* extends along the center of the track down the incline and through the cradle to a 30-in. pulley, then up to a 36-in. pulley at the track level, and has its end attached to the transfer car. The back-haul cable *B* also extends down the center of the track, but leads up the deck of the cradle and is attached to the car. Both cables are carried on 6-in. idler pulleys spaced about 10 ft. apart. The cradle cable *C* leads from the drum down one side of the incline and crosses through the upper end of the cradle. It returns along the opposite side of the incline, and its end is anchored at the trestle carrying the overhead supply track. Normally the cradle will be held in position by a ratchet brake on the first pair of wheels.

The transfer car has an end-dumping body pivoted to bearings on the frame. It stands under the hopper to receive its load, and is then hauled along the incline and

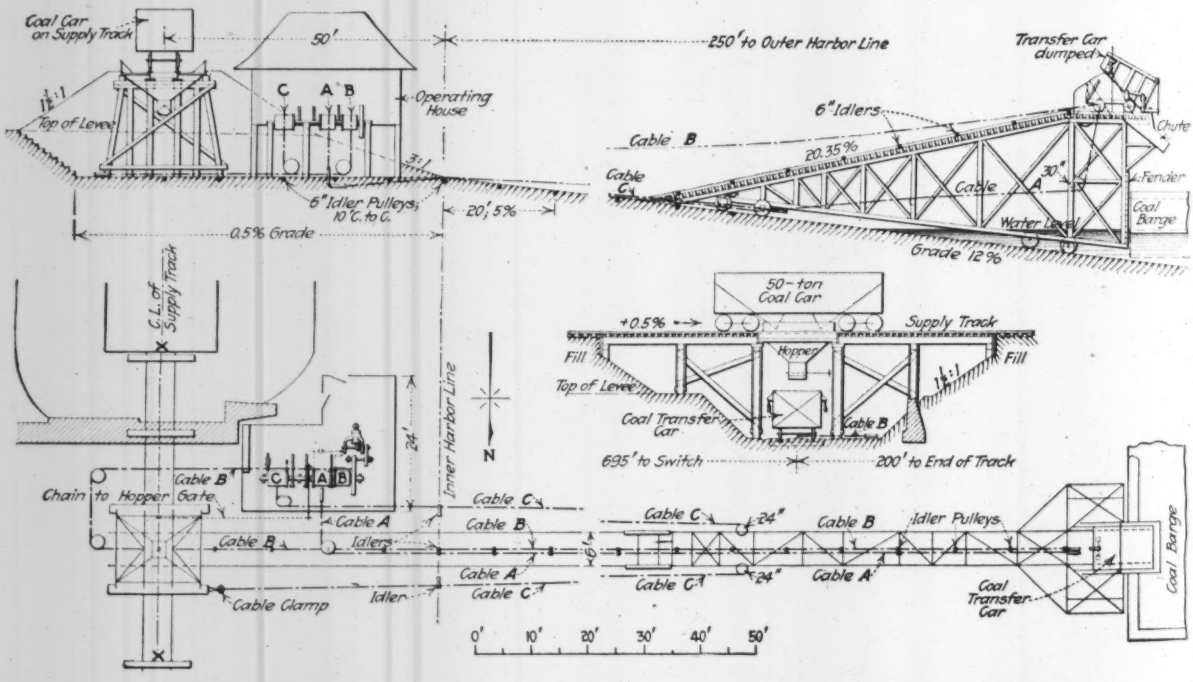


FIG. 1. COAL-SHIPPING STATION FOR LOADING RIVER BARGES AT EAST ST. LOUIS, ILL.

cradle. At the dumping platform, lugs on the side of the body engage with inclined guide rails, by which the body is raised at the rear (pivoting on bearings), while the forward end drops and the end door swings open, giving free discharge of the coal into the delivery chute. As the car is hauled back, the body drops into place on the underframe and is latched. The speed is variable, up to about 300 ft. per min. for the car both loaded and empty.

The cradle, transfer car and operating machinery have been built by the Weller Manufacturing Co., of Chicago. The coal shipping plant is for the Wiggins Ferry Co. The construction of the yard, approaches and tracks is being done by the latter's forces. The design and construction are under the direction of H. J. Pfeifer, Engineer of Maintenance-of-Way of the Terminal Railroad Association of St. Louis. The cost will be about $11,500 exclusive of the embankment for the supply track.

Experiments in Checking the Slides at Panama

The *Canal Record* of Jan. 5 publishes a special supplement containing an illustrated paper by Gen. George W. Goethals describing the Panama Canal slides and the methods adopted for dealing with them. For the most part the paper covers the same ground as that contributed by General Goethals to *Engineering News* of Nov. 25, although the subject is discussed in greater detail.

A notable feature of the article in the *Record* is a review of some of the experiments that have been made in attempting to check the progress of the slides. There is a more or less prevalent idea in the United States that the engineers on the isthmus have paid no attention whatever to any plan for stopping the slides other than to excavate the material as fast as it slips down into the prism. It is worth putting on record, therefore, that a number of methods for checking the slides, suggested by engineers, geologists and others, have been actually tried at the isthmus.

We append an extract from General Goethals' paper in the *Canal Record* summarizing these various experiments:

The various slides began without any warning, and there were no means of determining the extent to which they would proceed. Various attempts were made to check them, but all without success. There is no question that the excessive rains were responsible in a great measure for most of them and for the difficulties that resulted because of them; yet some of the most troublesome ones occurred during the dry season. Drainage proved ineffective. The rains, which cover a period averaging nine months of the year, so thoroughly saturate the ground that, though the surface may be dried out by the wind and sun during the remaining three months, the ground water remains. Because of the great depth of the cutting subsurface drainage could not reach the ground water sufficiently deep to be effective, even if the excessive cost involved warranted such a procedure. It has been suggested that artificial heat be applied through pipes, but the cost precluded such a method of relieving the situation; furthermore, the relief would be temporary.

Planting the slopes with grasses and vegetation prevents, to a certain extent, the erosion that follows some of the heavy downpours, but even in places where this has been done the results anticipated were not secured. The trees that have been standing on the banks for years slide down, standing erect in their normal positions, with slides of the second class and in the movements that take place subsequent to the "breaks."

Piling was tried with the hope that with the ends of the piles in firm ground the loose or moving portion might be retained in place. This also proved a failure, and along some portions of the banks are now seen piles projecting at various angles and at different elevations, though originally the piles were driven vertically and they were properly aligned.

Where the moving mass was clayey material loosened up by the movement and by the rains, a covering of heavy riprap was resorted to with the hope that their weight would carry the pieces of stone through the mass to the solid ground below and thus check, if not stop, the movement; much of this riprap was subsequently removed from the prism by the shovels.

It was believed that blasting was in some measure responsible for the slides, on the theory that the shaking up of the banks, caused by the blast, destroyed the cohesion of the particles in the banks, resulting in their breaking down, so that steps were taken to reduce the depth of the holes and the amount of explosive used, in order to lessen, if not remove, any source of trouble on this account.

It was learned that in experimenting with clays for the manufacture of pottery the Bureau of Standards has discovered a means of removing the slipperiness from the clays by inoculating the soils with a simple and inexpensive solution. With the hope that some such method of preventing the slides might prove effective with the soils on the isthmus, samples were sent for experimental purposes along these lines, but it appears that these clays are of an entirely different character, and no method of treatment has yet been evolved to secure the results desired.

The construction of retaining walls to withhold the moving masses was not possible, for access to the sides of the prism where the walls belonged could not be had. When access was possible, the movement had ceased, there was no evidence of any further movement and the desirability of or necessity for walls no longer existed.

Some of the sandstones and shales in the cut when exposed to the air disintegrate, but harden when kept constantly wet. Where disintegration occurred, the resulting soil would grow grasses and vegetation, and steps were taken to protect the slopes and the underlying material in this way, assisting nature to some extent in a country where vegetable growth springs up and expands rapidly.

Experiments were made with cement covering applied to the banks by the cement gun and by concrete held in place by rods embedded in the rock; neither proved successful, and they were abandoned.

When the use of concrete proved a failure, the geologist thought that experiment might develop a solution which, applied to the face of the sandstones and shales, would combine chemically with the substances in these rocks, so as to form a coating of glass. Experiments were made, but no satisfactory solution obtained.

With the breaks, except those which occurred in the vicinity of La Pita Point, lightening the banks, where this could be done, secured good results, as did also the sluicing of the upper portions of the hills around Cucaracha slide into the valley on the opposite side of the hills from the prism; but in all other cases the only effectual method found was to allow the material to enter the cut and remove it. This procedure has resulted in bringing all the slides to a state of rest, and with the exception of those now active none of them has given any trouble since, for there has been no movement of any kind in any of them after all the material that was in motion had been removed or come naturally to rest.

It is certain that the troubles are due to the failure of underlying strata, because these were unable to bear the weight that the banks brought upon them. Under the circumstances it is difficult to understand the impression that has gained credence in some quarters that a sea-level canal would have avoided the difficulties encountered, since the cutting would have been through the same material, but at least 80 ft. deeper.

It is also certain that nothing can stop the movements now in progress until the angle of repose is reached for the materials under the conditions that exist and that this can be reached only by removing the excess amount of material. If experience counts for aught, then that gained in the handling of the slides and the breaks that have occurred along the line of the canal leaves no doubt that the means adopted and now in use will effect a cure in the slides that now close the canal; furthermore, that when cured, no further troubles need be anticipated from slides in this locality.

Municipal Ownership of water-works, electric light and power plants in Kansas is thus summarized by "Kansas Municipalities": Of 206 water-works in the state, no less than 191, or 92.71%, are under public ownership. Of 166 electric light and power plants 88, or 53%, are municipally owned. There are in Ohio 110 and in Michigan and Minnesota 101 each of municipal lighting plants, but the percentage of municipal ownership is not as high in these states as in Iowa.

Field and Office

Building Concrete Piers for Sciotoville Bridge

The plant layout for the substructure work on the Chesapeake & Ohio Northern Ry. bridge over the Ohio River at Sciotoville, Ohio, was dictated by the necessity of getting sand and gravel in by river. A total of some 27,000 cu.yd. concrete and 850,000 lb. reinforcing steel was involved. The work comprised three river piers and twenty-four land piers.

On each bank an incline with narrow-gage track was erected, leading from low water to 40 ft. above the top of the bank (see layout plan, Fig. 2). The Ohio incline, the longer of the two, led up from a crib supporting a revolving or "whirler" derrick.[1] The Kentucky incline was served by a derrick boat. The engines of the inclines were of the direct-hoist type with large drums. All parts of both inclines worked satisfactorily throughout the season and a half that the work required.

The sand and gravel for concrete were obtained from a river bar a few miles below the site by means of a ladder dredge on which the sand and gravel were washed and screened. Flatboats delivered the material to the inclines, where it was unloaded with clamshell buckets.

At the top of the incline the skip dumped just below the head pulley and directly over a partition separating the sand and the gravel, there being a tilting apron on top of the partition, under the skip. The gravel used ran from ¾ to 3 in., undersize and oversize being wasted.

Materials for 8,000 cu.yd. of concrete were delivered to each bank, including that for the Ohio shore pier. All cement came in via the Norfolk & Western Ry. on the Ohio side. Cement for the Kentucky (south) viaduct, the Kentucky shore pier and the center pier was sent down the Ohio incline to flatboats, that for the Kentucky viaduct going up the Kentucky incline and that for the two larger piers going to the floating mixer plant.

The work was simplified by unusually good foundation conditions. In the river, rock was within easy reach. This was an important advantage, as on account of the structural type of the river span—two 775-ft. continuous truss-spans resting on three piers—it was necessary that there be no settlement of these piers.

The middle, or river, pier, which is the largest pier of this bridge, measuring 18x63 ft. under the coping and rising 96 ft. above low water, was built by means of a cofferdam. Work on this pier started late in October, 1914. A double-wall box coffer-dam was built, 79x127 ft. outside and 14 ft. high. At this spot the elevation of the river bottom, of practically bare rock and fairly level, was at low-water level. The 10-ft. space between the walls of the coffer-dam was filled with sand and small gravel, dredged from the river near-by, and decked over with 2-in. plank.

On account of the great rises of the river it was known from the beginning that the coffer-dam would be sub-

[1]See "Engineering News," Sept. 9, 1915, p. 506, for description and drawing of the Dravo "whirler" derrick.

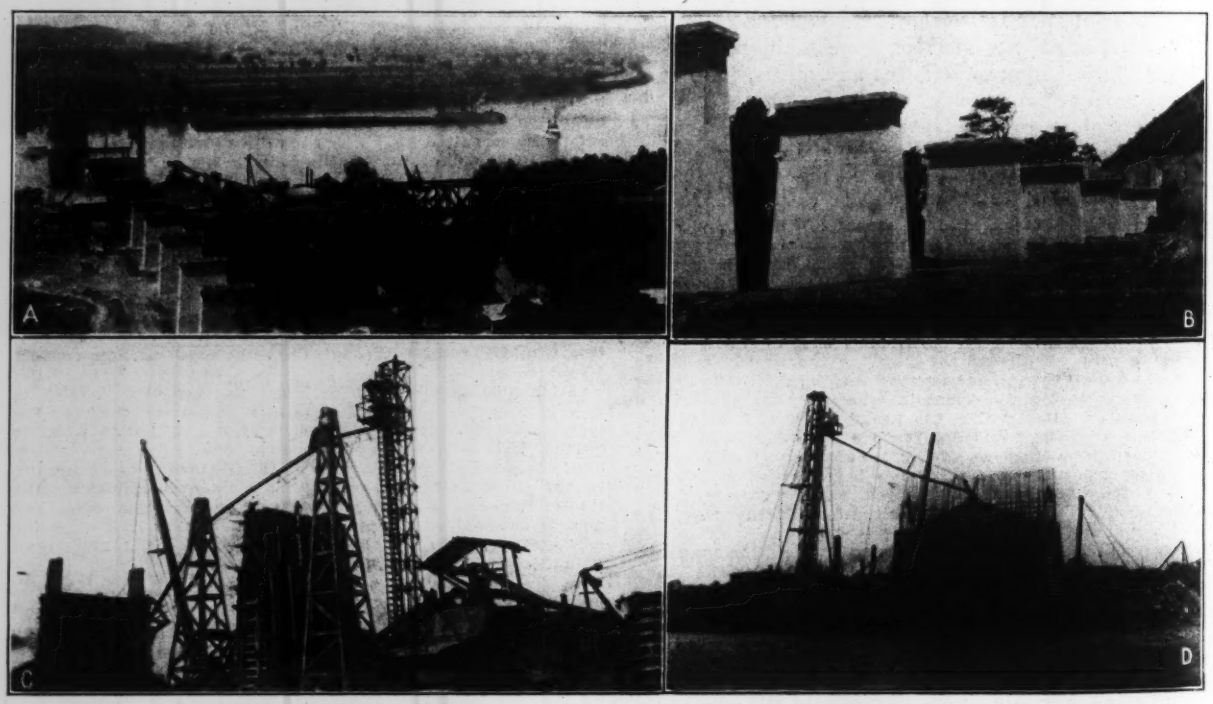


FIG. 1. VIEWS OF SCIOTOVILLE BRIDGE PIER CONSTRUCTION

A—Looking south along line of bridge; Ohio viaduct, north shore pier and center pier in place. B—Piers of Ohio viaduct. C—Spouting concrete to pier of Kentucky viaduct. D—Floating plant spouting to center pier

merged at all stages above a moderately high river. It was in fact submerged continuously for four months, but in the spring was found not to have moved, and practically no fill was washed out.

Inside the coffer-dam, open excavation was carried on by means of derrick boats to a depth of 10 ft. into the rock all over. The size of the footing is 40x85 ft. One No. 4 Emerson pump was used to keep the coffer-dam dry; it did not run at full capacity. Excavation was 30% completed on Dec. 4, 1914, when work stopped for the season on this pier. Work was resumed Mar. 29, 1915, the river being unusually low for this time of the year. The footing course was poured April 8.

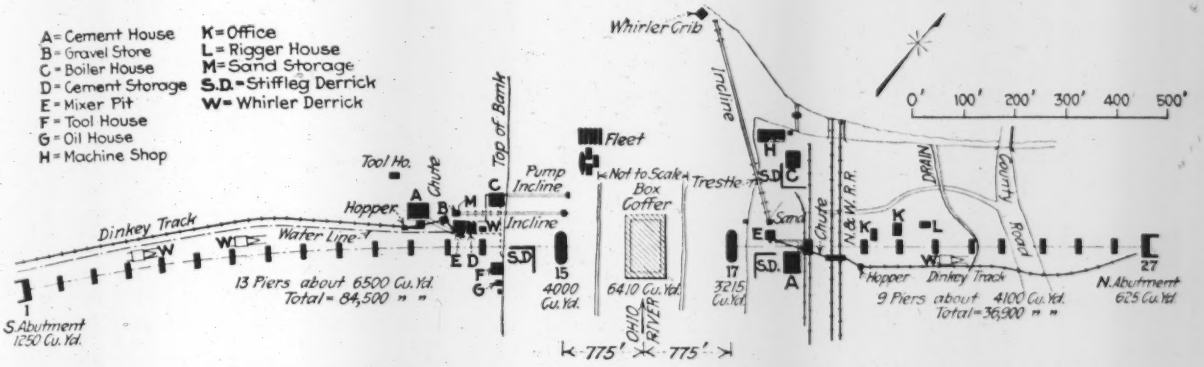


FIG. 2. DRAVO CONTRACTING CO.'S PLANT LAYOUT FOR SUBSTRUCTURE WORK ON SCIOTOVILLE BRIDGES

The Ohio shore pier was built in open excavation 24x70 ft., extending 8 ft. into the rock and high enough up the bank to be dry at ordinary river stages.

The Kentucky shore pier was sunk by open dredging through wells. It was so designed that it could have been converted into a pneumatic caisson at any time if it had become necessary. The pier cylinder is 24x70 ft., with semicircular ends. Excavation with orangepeel bucket, through four wells, was carried on with working chamber and wells flooded, after the first 5 ft. until gravel was reached. The pier went through 25 ft. of fine sand and 2 ft. of coarse gravel, and the final excavation (in the dry) was carried an average of 2 ft. into the rock. The sinking was successfully done in 16 working days for excavating only. The cutting edge was sealed and the working chamber kept dry while filling with concrete. The river was up around this pier during practically the whole period of its construction. The top of the pier cylinder was 28 ft. above the cutting edge, the working chamber was 6 ft. high, and the wells were 7 ft. 3 in. in diameter.

Both shore piers are 12x67 ft. under the coping. The Kentucky viaduct piers and abutment, with an average load of only 2½ tons per sq.ft., rest on a clean sand with depth of excavation averaging 8 ft., except the pier at the top of the bank, which goes down 25 ft. on account of possible scour of the bank.

Of the Ohio viaduct piers five rest on shale, while five are on rock, as is also the abutment. The second pier south of the county road was rather deep, on account of a gully. The abutment had to be stepped into the face of a hard shale cliff. Otherwise the excavation was very simple. The rock bottoms needed but little pointing to give level beds for the foundations.

The excavation for both viaducts was handled by whirler derricks on a track parallel to the line of piers. There were a 5-ton and a 10-ton whirler on the Kentucky side and a 5-ton on the Ohio side. Work at both shore piers was handled by a stiffleg derrick.

There were three concrete-mixing plants—a floating plant, and two fixed plants, one at the shore end of each incline. All these plants had 1-yd. mixer, 100-ft. tower, and chutes. The floating plant was served by a derrick boat which transferred cement and aggregate from flatboats direct to the mixer bins. With the aid of chutes this floating mixer poured all of the center pier and all except the coping of the Kentucky shore pier.

The chutes from the fixed plants were able to deliver concrete direct to the Ohio shore pier and to one pier of the Ohio viaduct and several of the Kentucky viaduct. For the remaining piers and abutments of the two viaducts the chutes spouted the concrete into hoppers, and from there it was carried by bottom-dump buckets on a dinkey track along the line of the piers (between them and the whirlers) to the pier where needed. On the Ohio side the hopper sat on the north side of the Norfolk & Western Ry. tracks, the chute from the mixer tower running over the tracks on a temporary bridge.

The viaduct piers are all rectangular in plan; all except two are 5x26 ft. under the coping and average 40 ft. in height. Slab forms were used in sections 16 ft. high and, for the sides, long enough to extend by the ends. These slabs were very substantial; they as well as the end bulkheads were made with vertical ¾-in. sheeting, 2x10-in. horizontal studding and 8x8 rangers. The tie-rods were fitted with a sleeve-nut 1 to 2 in. back of the face. The coping is somewhat complicated, but a well-made form of two sides and end bulkheads was built. Only two of these forms with minor repairs lasted throughout the job.

On the large river piers some of the slabs and a few of the round end forms were used more than once, but most of the forms for these three piers, including all of the three copings, were built new for each pier. Below the coping the forms were in 16-ft. lifts.

About 150 cu.yd. of concrete was usually poured in 10 hr., but on several occasions 300 to 350 cu.yd. was poured in 10 hr.

The bridge and approaches were designed and are being built under the direction of Gustav Lindenthal, Consulting Engineer, for whom O. H. Ammann acts as Principal Assistant Engineer and R. T. Robinson as Resident Engineer. The Dravo Contracting Co., Pittsburgh, Penn., was the contractor for the substructure. The plant layout was made by J. J. Nolan, and the subsequent work was under J. Smith Miller as superintendent.

Concrete Coal Cylinders Built with Moving Forms

By Charles H. Higgins*

The new retail coal plant of O. H. Perry & Son, Inc., at the foot of East 19th St., New York City, consists of three parts: (1) The concrete coal pockets; (2) the coal-hoisting machinery, headhouse and appurtenances, and (3) the office-scale house. Each part was constructed under a separate contract by different contractors.

The concrete coal pockets consist of a group, or battery, of nine cylinders, each of 18 ft. in inside diameter, supported on a reinforced-concrete platform of columns, girders, beams and slabs, and two additional concrete cylinders of the same diameter and similarly supported, located alongside the main battery and intended as a re-screening plant. In general the supporting structure consists of four rows of reinforced-concrete columns 18 ft. apart, each row consisting of from seven to nine columns. These support a platform of beam and slab construction, intended to support the cylindrical pockets, at such an elevation as to give 12 ft. clear headroom above the street level. Between each of the four rows of columns there is a driveway.

The sea wall affords a foundation for the row of columns nearest the river, and for each of the other three rows a foundation was provided consisting of wooden piles driven to refusal, cut off below water level and each row capped with a concrete wall and footing extending to within 2 ft. of the yard level and reinforced with steel near the top.

After the completion of the platform circular forms were erected for all nine cylinders, to a height of 4 ft., the plan being to cast the cylinders in a continuous operation by moving the forms up steadily as the concrete was poured. This is the type of construction developed in the West for grain bins, and it has recently been extended to coal pockets.

The walls of the cylinders were to be of concrete 7 in. thick, reinforced with 1-in. and ¼-in. steel bands spaced at varying intervals from the bottom to the top. The forms were constructed 4 ft. high and (for each cylinder) consisted of two outer and inner rings, each ring being made up of three thicknesses of 2-in. lumber sawed to the proper circumference and bolted together to break joints. Each pair of rings (the outer and the inner) was lagged with 2-in. lumber 4 ft. long, the ribs being located 2½ ft. apart, vertically and the lagging projecting correspondingly, as indicated in Fig. 2.

Between the outer and inner forms of each cylinder there were placed at regular intervals, in a vertical position, six 1-in. round steel bars, with the lower end on the platform. The rods were about 20 ft. long, and each extended through a heavy yoke made up of structural-steel members. Each yoke extended across the inner and outer form with a down-standing leg of angle iron 4 ft. long. Each vertical rod extended through a yoke, while a screw jack seated on each yoke and gripping each rod afforded means of lifting the forms by turning the jacks, which reacted on the rods.

A conventional concrete-hoisting tower some 70 ft. high had been erected at one side, with a batch mixer located in a pit at the foot.

The plan was to work continuously, concrete being poured in all cylinders and the band reinforcement being placed at the proper intervals as the concrete rose. The band reinforcement was secured to the vertical rods by wires. While some workmen poured concrete, others turned the jacks and raised the forms. Great care was taken to give each jack the same turn and to keep all forms moving together and plumb. The plan was to pour concrete continuously for two shifts of 8 hr. in each 24 hr., with an interval of 4 hr. between each shift, during which time the forms were kept moving by two men, who walked back and forth, giving each jack a turn. The work proceeded at the rate of about 6 ft. in each 24 hr., the forms, wheeling platforms, stanchions to carry the electric lights, etc., being carried up continuously by the movement of the jacks. At night the work was lighted by electric lamps hung from timbers.

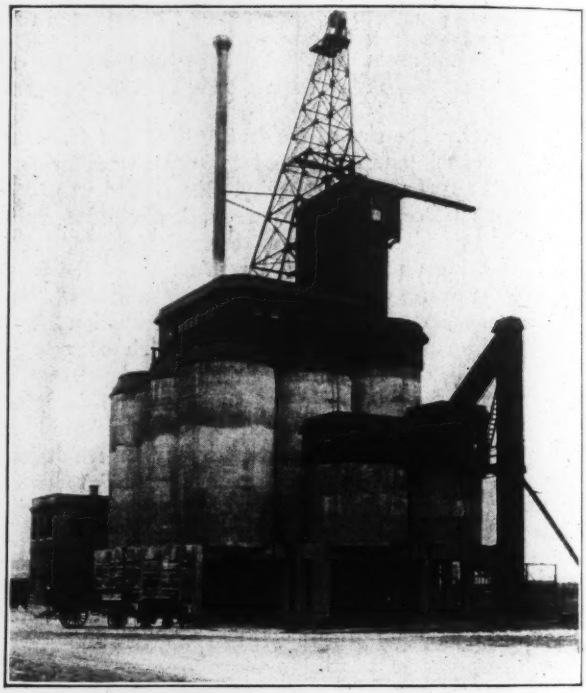
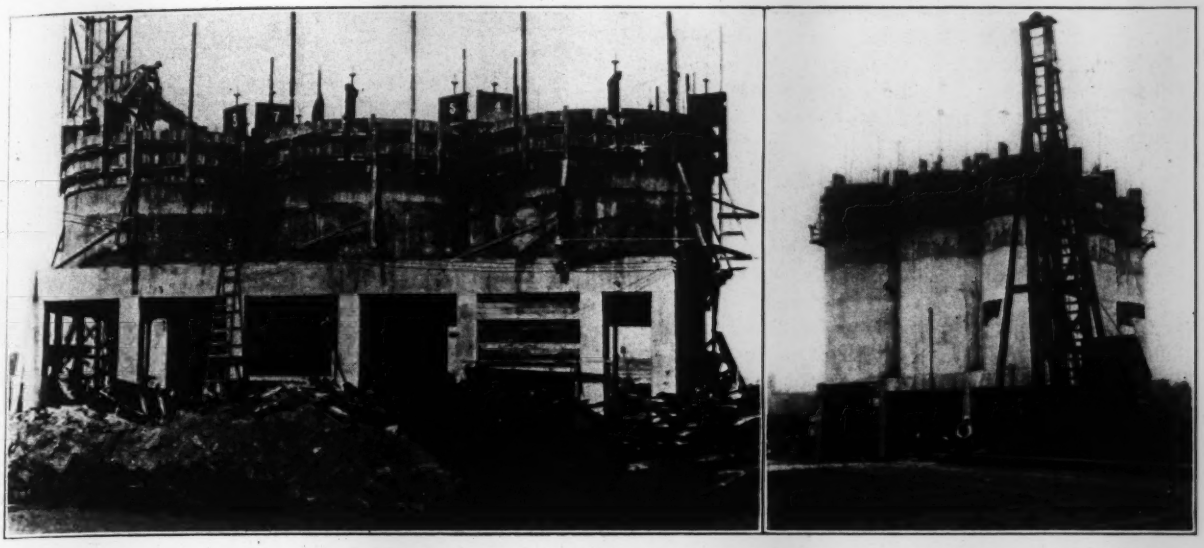


FIG. 1. COAL-STORAGE CYLINDERS BUILT WITH MOVING FORMS

In Fig. 3, which shows the work nearing completion, there is visible at the top a hanging scaffold, which was suspended from the outside of the forms and carried up with them. On this scaffold two men pointed up, rubbed and roughly finished the exposed surface. A high degree of finish was not aimed at, nor would it be possible by this means, as there is always a certain amount of cement-bearing water running down over the finished work.

The work progressed nicely, with no serious mishaps and with few difficulties. The most serious of those encountered may be seen in Fig. 3. The "prison windows" in this view are actually what is known as "pulls" and result from the concrete sticking to the forms and being pulled up as they advance, the reinforcement being left exposed to view. In general these were stopped at their very beginning by the alertness of the superintendent in charge; but in the cases seen in the photograph they progressed to such an extent that it was

*Consulting Engineer, 30 Church St., New York City.



FIGS. 2 AND 3. TWO VIEWS OF THE CONSTRUCTION; NOTE "PRISON-WINDOW" EFFECT WHERE FORMS STUCK

necessary to go on up, leaving an opening behind, which was later filled in by means of stationary forms and grouting. Very good results were obtained in repairing these openings, but the process is expensive, tedious and altogether to be avoided. It is probable that the more serious of these pulls resulted from the outside form at the critical spot not remaining perfectly vertical, the lower not moving in and pinching the concrete while the latter was still soft.

There were some delays in the program, the first resulting from an almost torrential rainstorm that drove the men off the work, the second, and more serious one, from the failure to deliver anchor bolts on time. As these anchor bolts—intended to hold the steel tower and machinery—had to be set in the concrete of the cylinders, it became necessary to stop work for a day or two. In each case, however, the forms were moved again successfully, and no more serious damage than a slight ridge resulted.

The concrete set nicely and the work, which was finished in the summer, now appears sound and serviceable, after coal has been in the pockets for some weeks.

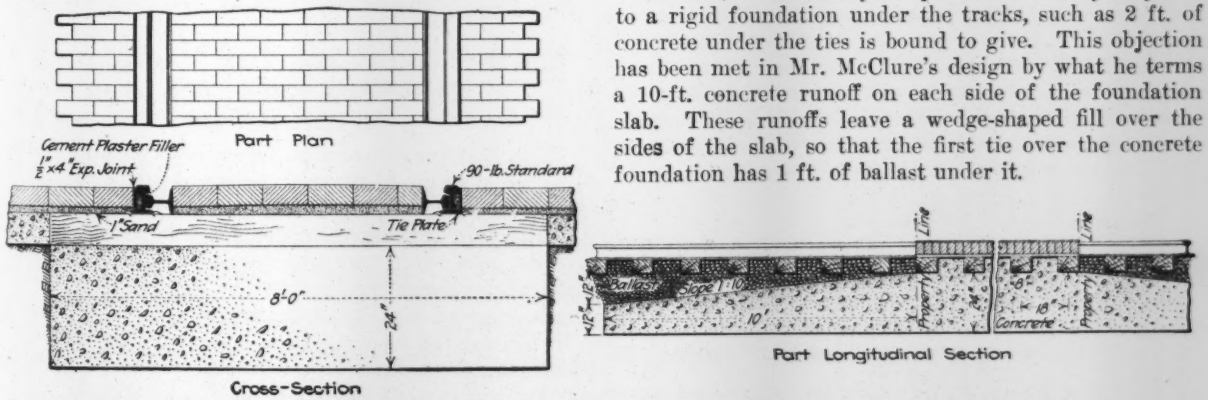
The construction of the rescreening pockets on the same principle was not determined upon until the main cylinders were reaching completion. Otherwise, they might have been carried up with the main cylinders. As it was, part of the forms taken off the main cylinders were used and the lower rescreening cylinders similarly constructed, for even in the case of two cylinders only 15 ft. high it appeared from the estimates that economy would result from the use of moving forms.

Ralph H. Perry was interested in the construction from the point of view of the owner. The writer was consulting engineer, preparing the plans, letting the contracts and supervising the construction. The General Contracting Co. built the foundations and the concrete coal pockets. The office-scale house was erected by Hugh Getty, Inc., the Fairbanks Co. furnishing the scales.

Railway-Crossing Pavement with Wedge Runoff

The city engineer of Oklahoma, Okla., Guy V. McClure, after a thorough trial has adopted as standard for railway street crossings the type of pavement construction shown in the accompanying illustration. Mr. McClure is a former railway construction engineer, and he has endeavored to design a crossing that will be satisfactory from the railway as well as from the pavement point of view.

Heretofore the railway companies have always objected to a rigid foundation under the tracks, such as 2 ft. of concrete under the ties is bound to give. This objection has been met in Mr. McClure's design by what he terms a 10-ft. concrete runoff on each side of the foundation slab. These runoffs leave a wedge-shaped fill over the sides of the slab, so that the first tie over the concrete foundation has 1 ft. of ballast under it.



STANDARD CONSTRUCTION FOR PAVEMENT OVER RAILWAY CROSSINGS, OKLAHOMA CITY, OKLA.

The effect of the sloped approach slab, with ties ballasted from 1 ft. down to nothing, is to take away the shock that would otherwise result in passing from the ballasted roadbed to the solid concrete-pavement foundation. This type of construction has been found to work satisfactorily even under the heaviest railway loadings.

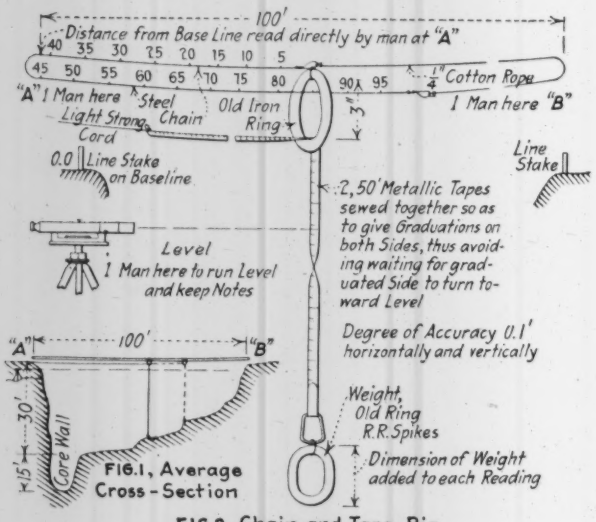
These crossings are installed by municipal day labor for about $5.40 per lin.ft. of track, including 10% for supervision and overhead charges, but not including the track itself, which of course is furnished by the railway company. The railways have turned over all construction work to the city. The cost of materials is: Sand, $1.50 per cu.yd.; stone, $1.60 per cu.yd.; cement, $1.50 per bbl.; paving brick, $16 per M. This method of construction requires that the track be out of service 72 hr.

Cross-Sectioning Device on Dam Excavation

By J. O. P. Coulling*

In excavation work for the new Loch Raven dam of the Baltimore Water Department the average cross-section was as shown in Fig. 1. To take a fairly accurate section by means of the ordinary rod-and-level method was laborious and dangerous. An easier method was therefore worked out and used successfully.

With an old 100-ft. steel chain and 100 ft. of ¼-in. cotton rope an endless chain was made, as shown in Fig. 2. At the zero end of the 100-ft. chain a 3-in. iron ring


100'
Distance from Base Line read directly by man at "A"
40 35 30 25 20 15 10 5
45 50 55 60 65 70 75 80 90 95
¼" Cotton Rope
"A" 1 Man here
Steel Chain
Old Iron Ring
1 Man here "B"
Light Strong Cord
0.0 Line Stake on Baseline
Line Stake
2, 50' Metallic Tapes sewed together so as to give Graduations on both Sides, thus avoiding waiting for graduated Side to turn toward Level
Level
1 Man here to run Level and keep Notes
Degree of Accuracy 0.1' horizontally and vertically
"A"
100'
"B"
30'
15'
Core Wall
Weight, Old Ring R.R. Spikes
Dimension of Weight added to each Reading
FIG. 1, Average Cross-Section
FIG. 2, Chain and Tape Rig

FIGS. 1 AND 2. CROSS-SECTIONING RIG USED AT LOCH RAVEN DAM

was fastened, through which a double 50-ft. metallic tape (two tapes sewed together so as to give graduations on both sides) weighted at the zero end was passed. A light strong cord was tied to the 50-ft. end of the tape.

Having set stakes on both sides of the excavation on the lines of the sections required and stretched the chain across, the wye level was set up a little lower than the chain. Then starting with the zero end of the chain at point *A*, the ring was pulled out over the section, and at the breaks the metallic tape was let down until the weight on the end touched the excavated surface. At such points the horizontal distance from the base line was read directly from the chain by the man at *A*. The original surface on both sides of the excavation was about the same elevation for each section, and no correction had to be made to obtain the horizontal distance. The rod reading was taken from the metallic tape. Readings as high as 40 ft. were secured. When one section was complete, the next was taken backward—that is, from *B* to *A*—the horizontal distances still being read by the man at *A*. The chief of party (at the level) and a man at *A* and *B* were sufficient to take the sections, although much better time was made with an additional man at the instrument.

*2431 East Federal St., Baltimore, Md. Formerly Assistant Engineer, Baltimore Water Department, on Loch Raven dam construction.

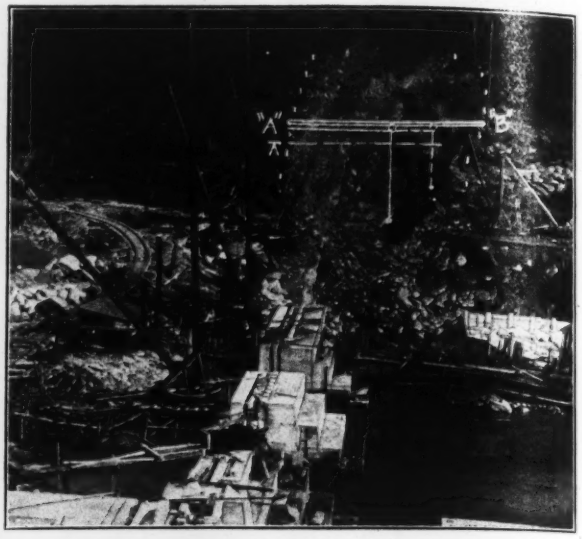


FIG. 3. LOCH RAVEN DAM UNDER CONSTRUCTION
White lines are drawn to show how cross-sectioning device was used

Sections taken in this manner were correct within 0.1 ft. horizontally and vertically, and a great deal of hard climbing and wasted time in making numerous set-ups was avoided. The men at *A* and *B*, however, found the narrow chain and small rope hard on their hands and were compelled to wear heavy gloves.

The view, Fig. 3, shows the character of the excavation. The only possible way to cross the nearly perpendicular rock face in the background was by a rope fastened to one of the trees above. The possibility of the rope breaking or coming unfastened, and the chance of starting loose material, would have made this extremely dangerous.

New Formulas for Weights of Girders and Trusses

By A. G. Hayden*

The following formulas for weight of girders and trusses in terms of the span length, depth, load and allowable stress have been found useful by the writer, particularly in making estimates for comparative designs of the same bridge, without the necessity of determining the girder-flange sections or the sections of the truss members and making estimates from the stress sheets. In general the formulas have been found as reliable as estimates from stress sheets in cases where the percent-

*Bridge Designer, New York State Barge Canal, Albany, N. Y.

...e of details is assumed. In comparison with the detailed estimates of many girders and trusses, the results obtained from the formulas have been found to be close, except in cases of inconsistent design.

It must be observed that difference in office practice as to details may require modification of the constants in the formulas, and that in any case marked peculiarity of design, particularly of trusses, may make the formulas inapplicable if very close results are desired.

Derivation of Plate-Girder Formulas

The average gross flange area for plate girders of variable flange section is about equal to the net area at the point of maximum moment. Details (stiffeners with their fillers, splice plates, rivets, etc.) are assumed to be 30% of the weight of the main section (flanges and web).

Let

M_t = Maximum total bending moment, including that due to the weight of the girder, ft.-lb.;
L = Span length, ft.;
f = Allowable stress, lb. per sq.in.;
h = Depth of girder, ft.;
t = Thickness of web, in.

If one-eighth the gross area of the web is counted as effective flange area, the total weight of the girder is, in pounds:

$$W = \left(\frac{2M_t}{fh} + \tfrac{3}{4} \times 12ht\right) \times 3.4 \times 1.3L = \frac{9M_tL}{fh} + 40htL \quad (1)$$

If no web area is counted as effective flange area, then

$$W = \left(\frac{2M_t}{fh} + 12ht\right) \times 3.4 \times 1.3L = \frac{9M_tL}{fh} + 53htL \quad (2)$$

The thickness of the web is usually required by specifications to be not less than $^1/_{200}$ of the depth of the girder $\left(\frac{12h}{200}$, in inches$\right)$ nor, for girders less than 6 ft. deep, less than $\frac{3}{8}$ in. Substituting these values in equations (1) and (2), we have if one-eighth the web is counted:

$$W = \frac{9M_tL}{fh} + 2.4h^2L \quad (3)$$

with a minimum for $\frac{3}{8}$-in. web.

$$W = \frac{9M_tL}{fh} + 15hL \quad (4)$$

If no web area is counted as flange area, we get:

$$W = \frac{9M_tL}{fh} + 3.2h^2L \quad (5)$$

with a minimum for $\frac{3}{8}$-in. web,

$$W = \frac{9M_tL}{fh} + 20hL \quad (6)$$

In the foregoing formula M_t, as previously stated, is the total moment, including that due to the weight of the girder itself, which is not known beforehand. The formulas may be reduced to a form by which the weight of the girder may be determined from the moment due to the superimposed loads (live, impact and dead), not including the weight of the girder. For example, in formula (1) if the maximum moment, neglecting the weight of the girder, be called M, we have:

$$W = \frac{9L}{fh}\left(M + \frac{WL}{8}\right) + 40htL$$

Solving for W and simplifying, we have:

$$W = \frac{9ML + 40h^2tfL}{fh - 1.1L^2}$$

Likewise (2) reduces to

$$W = \frac{9ML + 53h^2tfL}{fh - 1.1L^2}$$

Substituting the proper values for t, or treating (3), (4), (5) and (6) in the same manner, we have for final results, if one-eighth the web is counted:

$$W = \frac{9ML + 2.4fLh^3}{fh - 1.1L^2} \quad (7)$$

with a minimum for $\frac{3}{8}$-in. web,

$$W = \frac{9ML + 15fLh^2}{fh - 1.1L^2} \quad (8)$$

If no web area is counted as flange area, we find:

$$W = \frac{9ML + 3.2fLh^3}{fh - 1.1L^2} \quad (9)$$

with a minimum for $\frac{3}{8}$-in. web,

$$W = \frac{9ML + 20fLh^2}{fh - 1.1L^2} \quad (10)$$

These formulas checked against the estimated weights of girders for widely different spans and loadings gave close results.

For girders of very light flange section (small span and light loading) the average gross flange area will be more than the net center area, the actual flange section being nearly constant throughout. The constants in the formulas may be modified according to the judgment of the estimator to give more accurate results for this case. Thus, the coefficient of ML in the equations may be made 10 instead of 9.

Formulas (3) to (6) will be found to be more convenient for use, as the operator can make a fair enough guess of the weight of the girder to avoid a second approximation by formula.

Approximate Truss Weights

The weight of a riveted truss may be roughly estimated by the following formula. The constants were fixed so as best to suit the estimated weights of a large number of bridge trusses designed for the New York State Barge Canal. The formula gives more accurate results than any general formula the writer has noticed.

Let

W = Total weight of truss, lb.;
L = Span length, ft.;
P = Total load (dead + live + impact) carried by the truss, including the (guessed) weight of the truss;
f = Allowable stress in tension.

Then

$$W = \frac{13PL}{f} + 40L$$

The preceding formula may be reduced to give the weight W in terms of the total superimposed load (live + impact + dead loads), not including the weight of the truss itself. If this superimposed load be called p, then

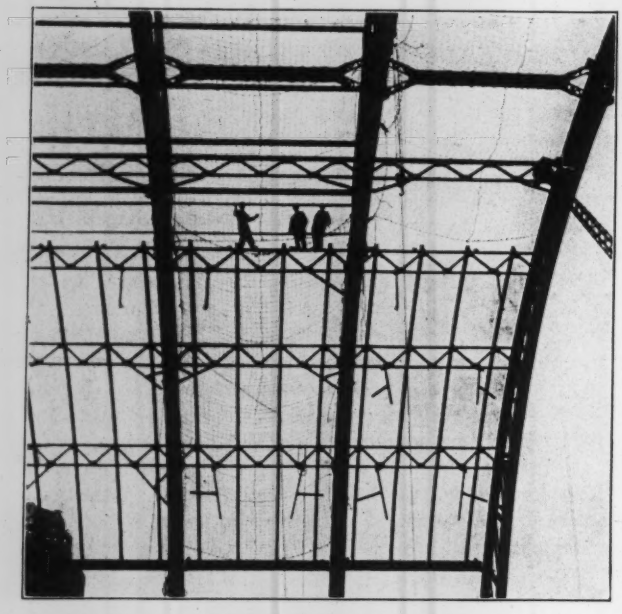
$$W = \frac{13pL}{f} + \frac{13WL}{f} + 40L$$

and, solving for W,

$$W = \frac{13pL + 40Lf}{f - 13L}$$

NOTES

Safety Netting on Steel-Erection Work—In the erection of the steel arch trusses for the recreation building on the Chicago municipal pier, rope nets were used to protect the erectors. During the previous erection of a similar structure a man narrowly escaped death when he fell off one of the trusses, which was struck and shaken by the boom of the erecting derrick. The state factory inspector called the attention of the contractors to the Illinois law requiring protection for men working in such dangerous positions, and on the recreation building the plan finally arrived at was to suspend netting beneath points where the men were working. This arrangement is shown in the accompanying photograph, which represents the removal of the nets after completion of the work. Steel-wire netting was not satisfactory, and the construction adopted was a modification of the rope netting used for the protection of circus performers. The nets are 20x40



NETTING FOR PROTECTION OF STEEL ERECTORS

ft., of ⅝-in. manila rope, with ¾-in. border. The mesh is 6x6 in. The borders are fitted with loops so that the nets can be readily attached to various points and shifted from place to place. When the job is completed, the nets are rolled up for storage or shipment.

State Highway Construction in Washington during the past year cost $1,222,000. With this sum 211 mi. of new state roads were completed and about 100 mi. of old road were straightened, widened and graveled. James Allen is State Highway Engineer of Washington.

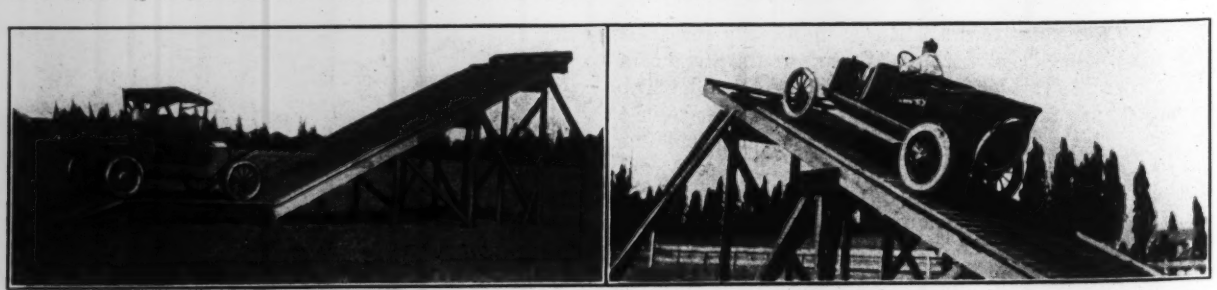
Elm-Tree Roots quite effectively choked 200 ft. of sewer in Concordia, Kan., as the accompanying view shows. The sewer was built in 1903 and was about 20 ft. distant from the trees. The roots almost completely filled the pipe, allowing only a small trickle to flow through. The largest roots were about ¼ in. in diameter. Needless to say, the joints

ELM-TREE ROOTS FROM A SEWER AT CONCORDIA, KAN.

were of very poor mortar. The photograph was taken in 1915, when a new sewerage system was installed by Black & Veatch, consulting engineers, Kansas City, Mo.

Jitney-Bus Traffic in Oakland, Calif., and the neighboring towns bordering San Francisco Bay received a severe setback when an ordinance was passed on Dec. 29, 1915, by the Oakland City Council prohibiting jitney cars from operating within the main business district of the city. The ordinance states that such action is absolutely necessary for the preservation of public safety, and this opinion is reported concurred in by most of the citizens of the trans-bay cities.

A Timber Incline used in demonstrating the hill power, or climbing ability, of a certain make of automobile, was recently built in Santa Rosa, Calif., by the Bailey-Comstock Co., engineers and contractors of that city. Mr. Comstock states that the stand was constructed with a roadway grade of 45° and rose at least 25 ft. from the ground at the highest point. It was heavy enough to withstand the shock of a large five-passenger machine hitting it with a running start and was designed to carry the weight of the machine and the kick from the driving wheels. A trap was built at the top to catch and hold the automobile in case anything went wrong with the clutch or in case the driver lost control while at the top. This trap was constructed by leaving the flooring planks off at the top of the stand and bracing with a heavy timber spiked across the ends of the joist. This opening would catch the front wheels of the car in case of accident. The demonstration consisted of speeding and spring tests, the latter being made by placing 4x4-in. timbers on the track and driving over them at a rate of 15 mi. per hr. There was a cooling test, consisting of a mile run without the fan, and last but not least came the climbing test on the demonstrating stand. From the price for constructing the incline the contractors deducted $5 for the privilege of placing a sign on it during the demonstration.



A STRONG TIMBER INCLINE BUILT FOR AN AUTOMOBILE HILL-CLIMBING DEMONSTRATION IN SANTA ROSA, CALIF.

Editorials

Rating the Qualifications for Success in Engineering

A circular letter has been mailed to all the members of the four great national engineering societies and, in addition, to the Society for the Promotion of Engineering Education, the American Chemical Society and the American Institute of Chemical Engineers, asking for information as to the qualifications that produce success in the engineering profession. The circular letter is signed by Prof. C. R. Mann on behalf of the Carnegie Foundation for the Advancement of Teaching, which is investigating engineering education in coöperation with the joint committee of engineering societies on that subject. The letter follows:

Last spring we consulted you on the subject of engineering education, concerning the qualities that you look for in young engineers and the general effectiveness of the engineering schools in producing such men as you want.

The replies that have been received mention many factors that are essential to the successful engineer. These essential factors fall naturally into the following six groups, which are named in the order of their importance as determined by their frequency of occurrence in the replies:

1. Character—integrity, responsibility, resourcefulness, initiative.
2. Judgment—common sense, scientific attitude, perspective.
3. Efficiency—thoroughness, accuracy, industry.
4. Understanding of men (executive ability).
5. Knowledge of the fundamentals of engineering science.
6. Technique of practice and of business.

If this is a correct definition of the essential characteristics of an engineer, it furnishes the necessary basis for a rational discussion of the effectiveness of the schools. It can also be used as a standard in developing staff organizations and curricula and in selecting efficient methods of instruction.

The best way in which engineers can aid the schools in solving problems in connection with engineering education is by defining the product which they desire. We therefore urge you to contribute to this important cause by numbering the foregoing six groups in the order of importance which you give them in judging the reasons for engineering success and in sizing up young men for employment or for promotion.

Any engineers who have not received this letter and who may be interested to aid in the investigation are invited to send their replies to Professor Mann.

At the recent annual meeting of the American Society of Civil Engineers Professor Mann reported progress in the investigation and read the list of qualifications enumerated and the ratings that have been awarded to them in the replies received from several thousand engineers. The rating was made on a scale of 100 and, according to the average of the ratings as stated by Professor Mann, the percentage assigned to the other grounds of qualities was so high that knowledge of the fundamentals of engineering science as a requirement of success was rated at only 13%.

This statement was greeted with tumultuous applause. When one attempts to analyze it carefully, however, it is perceived how unsafe may be deductions from such investigations as a guide to practical work in engineering education. In hiring young men for subordinate positions in an engineering organization "knowledge of the fundamentals of engineering science" by an applicant is rated by the employer at very much more than 13%.

It seems evident that those who, in answer to the questions sent out by the committee, have stated the qualifications that make for success in engineering have had chiefly in mind the qualifications that enable a man to achieve leadership in his profession rather than the qualifications that enable the average engineer to earn his living at daily routine work.

It is in the application of these indexes to success to the practical problems of technical education that error might most easily occur. Such personal qualifications as character, judgment, efficiency and understanding of men are to a very considerable extent inborn qualities. They may indeed be cultivated and improved or allowed to deteriorate, but they are on a wholly different basis from the knowledge of engineering science, which it must be the primary function of the engineering schools to impart, even though these schools may well at the same time do what they can to develop and improve those personal qualities of the student which are undoubtedly essential to the highest success.

Who Should Design Temporary Subway Timbering?

A chief feature of subway building as carried on so extensively in New York City is the temporary timbering for carrying the street surface and for retaining the slopes where necessary. This construction is not only omnipresent and therefore very extensive and costly, but is frequently intricate and of varied design to fit the special conditions continually and literally unearthed. Under the present manner of contract letting, the contractor designs the timbering, subject to approval by the Public Service Commission. Would it be more equitable to contractors and safer to the "man in the street" for the Public Service Commission to design the temporary timbering prior to the letting of the contract and have it bid on as a part of the contract? In "Secure Subway Supports," read at the regular meeting of the American Society of Civil Engineers on Feb. 16, the authors, A. B. Lueder and W. J. R. Wilson, have the following to say on this important subject:

It would be conducive to greater safety if the main features of the system of support to be used were designed before letting and bid upon as part of the contract. Systems have been designed piecemeal from the bottom up by combining the ideas of timbermen and other workmen, but the writers believe that the safety of city streets would be better guaranteed with a comprehensive design made by competent engineers. The authority and responsibility connected with the design of street supports and the inspection of them when placed or when shifted or removed should be in as few hands as possible.

This, like many questions, has two sides. Many contractors will hold, and perhaps with justice, that such a procedure would be unfair to them. There are many methods of timbering, and each contractor has to a certain extent his own pet system and an organization that is familiar with that particular system. Furthermore, he must select a method that will fit his equipment.

An efficient system of street supports is not one that is merely safe. Even though it were "absolutely safe," it might not be good timbering from the standpoint of economy of money and time, both of which are important elements in work of this size and character. Many of the ingenious designs of subway timbering have been evolved by contractors—or perhaps more properly, by the engineers employed by contractors—under financial stress. These men will claim that if a rigid system is laid down for them to follow, it will be one more obstacle that they must overcome in prosecuting this already difficult and even harrowing work.

Could a comparatively rigid method be followed? Special conditions of soil would make frequent departures from the system necessary. When confronted by unforeseen conditions, the contractor would have to ask permission of the Public Service Commission to use another method. In legitimate cases of this sort he would probably be told to go ahead and use the other method, but at his own risk. Thus the contractor would be responsible for a certain amount of the timbering anyway. This condition would create a division of responsibility, probably with the orthodox result.

Messrs. Lueder and Wilson believe, and rightly, that the systems of temporary supports should be designed by competent engineers. But contractors with organizations big enough to handle subway contracts employ competent engineers. Whether the systems are designed by competent engineers in the employ of the Public Service Commission or of the contractors should make little difference as long as those systems are approved by the commission. Approval by the commission is of course a necessity; although an equally important matter is that there should be continual inspection of the street supports by the commission.

A compromise between the two opinions would be for the commission to include in the specifications one or two alternate systems of supports, giving the contractor the privilege of submitting a design of his own. That it is difficult to estimate the cost of timbering is undeniable, and to have a definite system specified by the commission might be welcomed by some contractors.

A last question: Does the city or the Public Service Commission want to be responsible for the subway timbering?

Typhoid an Accident Under Compensation Law

Holding mill owners responsible for deaths from typhoid fever caused by faulty connections between polluted mill water-supplies and city water-supplies will hasten the day when this grave danger will be greatly reduced, if not quite eliminated. The exact method of pollution in the case mentioned below is not reported, but the usual method is a defect in the design or maintenance of the check valve that is supposed to prevent the polluted mill supply from getting into the city supply, while admitting a flow in the other direction.

The case in point is all the more interesting and important because it holds an employer liable for a typhoid death under a workmen's compensation law. The facts are these: A lumber mill at Eau Claire, Wis., discharged its sewage into the river at such a point as to pollute its boiler water-supply, the pipe for which was also connected with the city water-supply. An employee who drank this polluted water died from typhoid fever on July [illegible], 1911. A suit for damages was brought. On demurrer the Wisconsin Supreme Court decided that such a death was "proximately caused by accident" due "to his employment." (Vennen vs. New Dells Lumber Co., 15[illegible] N. W. Rep., 640, Oct. 26, 1915; also *Public Health Reports*, United States Public Health Service, Feb. 11, 1916.)

The court seems to have upheld the contention that the typhoid was caused by "the undesigned and unexpected occurrence of the presence of bacteria in the drinking water" supplied to the deceased, and that this was an "accident" under the meaning of the Wisconsin compensation law. One dissenting justice held that such an interpretation went far beyond the intent of the legislature, which he maintained had physical injuries, like blows, in mind.

The opinion of the majority seems sound. Why should not a workman's family be compensated for negligence that fills his system with millions of typhoid germs and causes death after a long and painful illness as well as for negligence that strikes him a swiftly fatal blow?

Power Plant for Government Buildings in Washington

A widespread campaign of newspaper publicity has recently been carried on to stop the construction of the new central plant to supply light, heat and power to Government buildings in Washington. It has been alleged that the engineers have so located the structure that it will be a hideous blot in the landscape and a permanent disfigurement of the plan for the beautification of the capital city. The architectural profession and its societies have been especially active in stirring up this opposition and have secured the support of prominent engineers.

Engineering News has taken pains to ascertain the facts concerning the project and the arguments made for and against it. Unimportant though the whole matter is in comparison with the profound events now stirring the world, certain questions of engineering interest are involved, as well as the question of national interest as to whether harm is threatened to the beauty of Washington.

It has long been realized that there would be great advantage if the group of Government buildings in Washington lying between the White House and the Potomac could be furnished with light, power and heat from one central station. The principal advantage to be gained is not the money saving in fuel and operating cost, although a central station does promise a large gain in this respect. Of much greater importance is the removal of the independent power plants now necessary in each of these buildings, thereby making it possible to do away with the hauling of coal and ashes to and from these buildings and, more important still, to put an end to the discharge of smoke, dust and gases from these individual power plants, which results in inevitable soiling and defacement of the building exteriors. This plan further makes it possible to remove the projecting chimneys from these Government buildings.

These obvious advantages of a central light, heat and power station led Congress in 1904 to direct an investigation of the matter by the late Bernard R. Green, then superintendent of the Congressional Library building and

engineer of eminence who was a leader in many projects for the beautification of Washington.

Mr. Green in his report strongly recommended the construction of such a central plant and suggested three different locations. Nothing further was done about the project for seven years, until in 1911 Secretary of the Treasury McVeagh appointed a committee of three engineers to report plans for such a plant. This committee reported plans for a plant to be built on that one of the three sites recommended by Mr. Green that was most remote from the buildings to be served. A special vote of thanks was passed by President Taft and his cabinet to this committee of engineers for its work.

On June 23, 1913, Congress passed an act to carry out the recommendations of the committee. The Secretary of the Treasury employed the firm of L. B. Stillwell & Co., engineers, of New York, to go over all that had previously been done, recommend any desirable changes and prepare plans and specifications for the work. The contract for the construction of the plant under these specifications was awarded on Dec. 24, 1915, to W. C. Cornell Co., of Washington and New York, and work is now in progress.

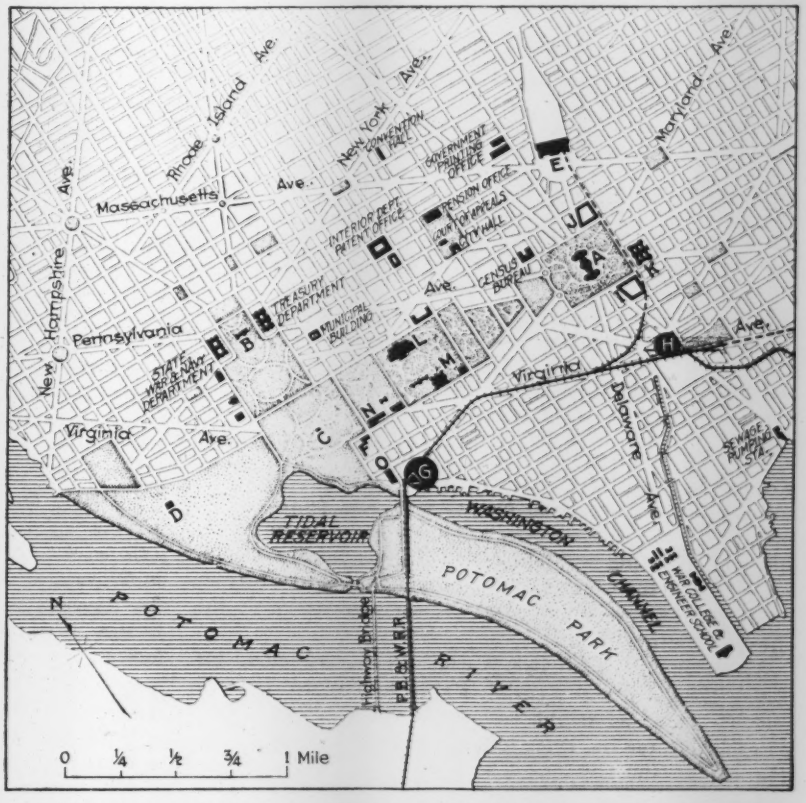
Under an executive order all plans for Government buildings in Washington must be submitted before execution to the Fine Arts Commission. Through an oversight (due to the fact that the detailed plans were prepared by an engineer outside the department) this was not done until after the contract had been awarded. When the plans were submitted, the Fine Arts Commission objected to the location of the plant on the site selected and also suggested various modifications in the design. The advice of the commission was not accepted by Secretary McAdoo, however, and the campaign to stop the work on the plant and open the whole matter anew appears to have been the direct result. The matter has been the subject of long and heated debate in the Senate, and on Feb. 15 a resolution was passed requesting the Secretary of the Treasury "to modify the plans for the work as far as possible, to better harmonize with the Polk Commission plans for the improvement of the commercial waterfront and with the general beauty and comfort of the City of Washington."

The opposition has centered chiefly on the proposed location of the plant, and the statement has been published that the plant is located on the Mall, where it will disfigure the great group of Government buildings planned for that space. The actual location of the plant, however, is on the Potomac River front behind the embankment of the railway approach to the Potomac River bridge. Much of the plant will be hidden from view from the Government buildings it serves and from Pennsylvania Ave. by this embankment. The plant is 1⅛ mi. distant from the White House; and while the tops of the stacks are at an elevation of 175 ft., all but their very tops are hidden from points of view near the White House by the roof of the great building of the Bureau of Engraving and Printing.

The accompanying sketch map of central Washington shows the location of the plant with respect to the principal Government buildings. It will be seen that the plant is located on the commercial waterfront of Washington; the piers for oyster-boat and excursion traffic are immediately adjacent.

In a statement furnished by Secretary McAdoo on Feb. 7 to the chairman of the Senate Committee on Appropriations the secretary urged the advantage of the site as follows:

> The selection of this particular site for the new power house was dictated by the ABC's of mechanical and electrical engineering. It is so placed as to be readily and cheaply served with coal both by rail and by water, and the ashes can be cheaply and readily removed by the same means. Condensing water in large volumes is close by and can be handled with the minimum expense for pumping. The plant is fairly within the center of gravity of its load. It is within 800 ft. of buildings which will take 50% of the steam generated and nearly 40% of the electric current. The plant is located on the commercial waterfront on property owned by the Government, and at the least conspicuous place in Washington for such a plant. Its construction will eliminate 17 existing smoke-stacks on Federal buildings now discharging smoke and gases. The plant will effect an annual saving in the supply to present buildings estimated at $177,000, and a further saving of $85,000 when additional buildings are supplied.



SKETCH MAP, SHOWING LOCATION OF PRINCIPAL GOVERNMENT BUILDINGS IN WASHINGTON AND SITE OF NEW CENTRAL HEATING PLANT

A—Capitol. B—White House. C—Washington Monument. D—Lincoln Memorial. E—Union Railway Station. F—Old Bureau of Engraving and Printing. G—Site of new central heating plant. H—Garfield Park. I—House office building. J—Senate office building. K—Library of Congress. L—New National Museum. M—Old National Museum. N—Department of Agriculture. O—New Bureau of Engraving and Printing

Those who oppose the present plant do not oppose the construction of a central power plant, but argue that a different site should have been selected. They take this position, not so much because of any injury to the view from the White House grounds or the Mall, as because of the park development on the opposite side of the Washington channel and along the river front to the west. They urge that the plant should be built as an addition to the present central power plant in Garfield Park, which supplies the buildings grouped around the Capitol.

The accompanying map will make clear to the reader the location of the two sites and of the principal Government buildings. It must be borne in mind in considering the question of relative location that the use of the plant to distribute steam is the chief factor to be considered. Were it merely a power plant, it could readily be removed to some more remote location; but the practical difficulties in transmitting steam long distances through tunnels under city streets are well known. The distance from Garfield Park to the more remote buildings on the Mall is nearly three miles.

In a resolution approved by the Council of the American Institute of Consulting Engineers opposing the site adopted the following appears:

> The construction of a power station, with its towering chimneys, upon this site will be a hideous object in the foreground of the beauties of Washington, upon which the nation is spending many millions of dollars.

It is a fair question to raise, Why should a power station be a "hideous object?" The "towering chimneys" are not essential. It makes only a trifling difference in the operating expense of the station to provide for draft enough so that the chimneys can be lowered to a level with the roof line. The power-station building itself may be as beautiful in its architecture as any other structure. A power station in Frankfort, Germany, with its fine architecture and stained-glass windows, is one of the show features of the city.

In the present condition of Washington's commercial waterfront, with its old wooden buildings, any power station would be an adornment. In the future development planned for that waterfront the power station can be made at least as beautiful as the piers and warehouses.

It has developed as a result of the agitation that the plans for the new power station are inadequate, having been made to fit a limited appropriation instead of planned to meet the actual needs, ornamental as well as economic. A radical revision of these plans would be to the nation's advantage, but a balance of all the arguments thus far presented indicates that no mistake was made in the choice of a site for the plant.

Letters to the Editor

Power Developments Along the New York Barge Canal

Sir—The articles in *Engineering News* of Jan. 20 and 27, on the operation of the New York State Barge Canal, make a few references to factory sites and power developments along this waterway and afford an opportunity of again pointing out how essential it is for the proper operation of the canal that the present legislative restrictions on the sale of canal power be removed with the greatest caution.

When the original Erie Canal was constructed, the canal commissioners granted rights to the use of such surplus water as might be created by the state dams, in exchange for the right to build those dams and utilize the fall in the stream. The canal itself was the best means of transportation then available, and these power sites attracted industries. In these early days 36 to 40 leases were granted, usually in perpetuity with a small annual rental and providing that the state could resume the use of the surplus water for navigation purposes at any time.

Most of these leases were canceled at an early day for nonuse, or nonpayment of rental. The total amount of remuneration that the state obtained through its sales of surplus waters on the Erie Canal for the 75 years down to 1910 inclusive aggregates only $191,572. The Kennedy & Hatch lease at Lockport and the lease now owned by Finch, Pruyn & Co. at the state feeder dam on the Hudson River above Glens Falls are still in force. In the case of both these leases the Supreme Court has recently held, as the writer understands it, that as long as the rental is paid, the state cannot cancel or revoke these leases. If the amount of surplus waters available at these sites is increased, then this additional water may be utilized as surplus water under the original leases without payment of additional rent.

As early as 1826 it was found that the drawing of surplus water from canal levels for power purposes interfered with navigation. Boatmen are wasteful of water on the canals. No matter how abundant the water-supply, the boatmen would succeed in using or wasting it all, if they were permitted. Their object, as a rule, is to facilitate the passage of boats by sluicing them through locks. They prefer to keep the canal levels at their maximum stages by wasting water, rather than to exercise greater care in navigation in order to avoid the possibility of running aground.

The menace to the canals from the draft of surplus waters was a primary cause of the revocation of most of the outstanding leases about 1840 and the refusal of the state canal commissioners to grant further leases. In 1870 a general investigation was made by the canal commissioners. It was reported that the use of surplus water for power was inimical to navigation and that there was no apparent remedy.

For the present, at least, no provision should be made that will allow surplus water power created by the Barge Canal to be used in such a manner as to cause the slightest interference with navigation. Indeed, if the development of surplus power from canal waters is carried out even in its fullest extent subject to this restriction, it will be found, first, that the maximum amount of power available (about 50,000 hp.) is considerably less

it appears to be the case from estimates based on ideal canal-operating conditions, and, furthermore, that the value of such power is as a rule considerably less than the value of an equal amount of otherwise equivalent power not subject to such uncertainties and restrictions.

There is about half a million horsepower of water power in use within the state at the present time, and an equal or greater amount is or can be obtained from the Niagara and St. Lawrence Rivers. This supply, with the possibility of a large increase in the power obtainable at privately owned power sites as a result of 24-hr. use of existing power and as a result of more efficient development and control of the streams by storage where feasible, makes it evident that the total amount of water power owned by the state is not a controlling factor as regards the market for water power in the state. The removal of the present legislative restriction on the sale of canal power should be deferred until the Barge Canal has been in actual operation long enough so that its operating conditions can be determined and the mistakes of 1826 avoided.

ROBERT E. HORTON,
Consulting Hydraulic Engineer.

57 North Pine Ave., Albany, N. Y., Jan. 30, 1916.

Why a Certain Grouted Brick Pavement Heaved

Sir—In your issue of Jan. 13, 1916, I noticed a letter signed by F. T. Gillett, Engineer Vincennes Traction Co., asking the readers of *Engineering News* if they could explain the failure of a certain grouted brick pavement laid between rails of a street-railway track in Vincennes, Ind. On this job the sand and cement were wet with a hose before the joints were grouted.

This is not a practical method to employ in freezing weather, as water will not run through the material, as it does in warm weather. The frost is given an opportunity to heave up the brick immediately. The rails allow no expansion of the pavement; and no matter how carefully they are spiked or bolted, the weight of cars going over them causes them to wave slightly, which greatly increases the tendency of the brick to hump. I have seen exactly this condition in New York City.

Some years ago I laid a driveway, using the same method that Mr. Gillett employed. This driveway was between two buildings. It was cold weather when the work was done, and within two weeks the pavement had heaved. I never recommend the use of this method in very cold weather.

EDWIN C. BRADLEY.

Graham Manor, West Haven, Conn., Jan. 23, 1916.

Pile Formula Misinterpreted

Sir—I noticed with considerable interest the article entitled "Pile Formula Overruled by Test," in your issue of Jan. 20, 1916. On reading the article I became very much impressed by the injustice meted out to the formula, the name of which was not stated, but which I presumed to be Wellington's, more commonly known as the *Engineering News* formula.

This formula, as everyone knows, gives the safe load immediately after driving. The writer of the article evidently must have overlooked that fact. The tests which he quotes as "overruling" the formula were taken during a period of 14 days, during which time the loads were increased from 5 tons to 35 tons. Previous tests have amply proved the fact that the effect of rest on a pile in certain kinds of material, and especially in a sticky clay such as this happened to be, is to increase the bearing power of the pile. Now, computation by the formula gives a safe load immediately after driving of 4 tons, which was approximately borne out by the test, showing that the formula was not in any way "overruled."

There is so little accurate information in regard to piledriving, because of the many varied conditions encountered, that it seems as though we should be very careful how we attempt to discredit a formula that has been of considerable use, especially when we are so careless as to put a wrong interpretation on such formula.

Ithaca, N. Y., Jan. 29, 1916. C. A. THOMPSON.

Business Methods of Patentees

Sir—When confronted with a demand for royalty on a reinforced-concrete retaining wall in place two years, a city water commission was advised by the writer, who designed the wall, that such a patent had no legal standing. This bears out your statement in *Engineering News*, Feb. 10, 1916, p. 259: "The average engineer is apt to take a snap view of the legality of the Bone patent." The writer's advice was not followed; a majority of the water board, having knowledge of the high cost of lawsuits, decided to pay up. They may have been helped to this decision by the fact that the patentee's demands were comparatively moderate and by sight of a photograph of the $18,000 check made out by the Chicago, Burlington & Quincy Railroad Co.

Whatever view the courts may take ultimately of the legality of the Bone patent, the writer believes that Mr. Bone's business methods do not measure up to present-day ethical standards. In 1908 F. A. Bone, signing himself General Manager of Concrete-Steel Retaining Wall Co., wrote a very instructive article in *Engineering News*, Mar. 26, p. 347, showing a saving of 50% and over by use of an L-shaped instead of a gravity wall. After having aroused in the reader such an appreciation of the L-shaped wall, it would have taken but a few strokes of the pen to mention patent No. 705,732, granted July 29, 1902. But the author's ink gave out, and the interested reader never guessed that this favorable economy must be somewhat reduced by the royalties due the patentee.

Descriptions of patented articles have their place in engineering literature, but the fact that the article is patented should not be left for the reader to guess. Compare the above methods with the article in *Engineering News*, July 2, 1914, on the Wheeler filter bottoms, which concludes with the statement that the patents had been applied for in 1913.

There is another phase of Mr. Bone's business methods—his reluctance to put himself forward until the structure is completed and in use. When the writer was called upon in 1910 to design several hundred feet of reservoir wall, he looked up Mr. Bone's article, made comparative estimates for local conditions and chose Mr. Bone's type. This contract was advertised in *Engineering News* in the fall of 1910, but not until four years later did the patentee come out with his bill for royalties. Innocent of the existence of such a patent, the writer had

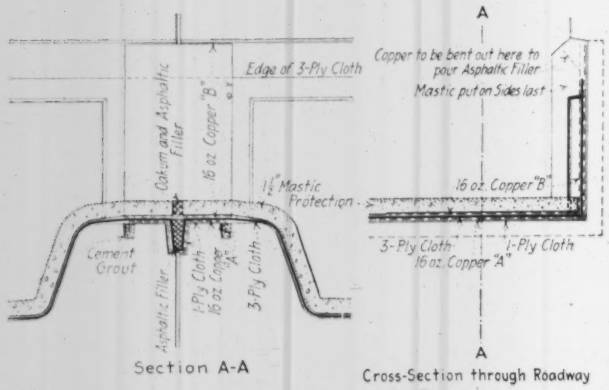
not included cost of royalties in his engineer's estimate. The patentee's demands put the designer in a very uncomfortable position before the lay members of the board of water commissioners; he felt himself under suspicion of stealing the design. This could have been easily avoided, had Mr. Bone started correspondence when the contract was advertised.

If. Mr. Bone's patents are legal, he has no reason for withholding his demands. Even counting in royalties, his type of retaining wall will often be found the most economical. Why does he not modernize his business methods? Then engineers will more cheerfully accede to his demands. BONEHEAD.

Feb. 12, 1916, New York City.

Waterproofing the Lackawanna Railroad's New Viaducts

Sir—My attention has been called to the fact that misleading, or at least incomplete, statements of the facts with reference to the method of waterproofing the Martins Creek and Tunkhannock viaducts, on the new line which the Lackawanna has just built west of Scranton, have been made by a firm advertising its waterproofing materials. Since the writer's name has been used in connection



DETAILS OF WATERPROOFING OF THE TUNKHANNOCK VIADUCT

with some of these advertisements and wholly without authority being given to the advertiser, he feels that it is essential at this time to make a short and concise statement of the facts, so that there need be no further misunderstanding as to the kind of materials used or how the work was done.

Both the structures referred to were treated alike, the same waterproofing materials being used in each case. The only material difference is the fact that the Martins Creek structure is a three-track viaduct, with two separate lines of drain pipes in the floor system, while the Tunkhannock is a two-track structure, with but one line of drain pipes.

The floor system over each main arch is divided into three parts by four transverse expansion joints—two adjacent to each pier and one at each of the quarter points of the span. The floor is drained by downspouts through all spandrel walls, excepting those at the two intermediate expansion joints, and the drainage is discharged into the openings between the two ribs of the main arch. The drainage is prevented from flowing over the expansion joints by dikes built across the floor.

The waterproofing proper was done by using three plies of Minwax saturated cotton fabric laid in Minwax hard waterproofing. The concrete was first mopped with the Minwax hard waterproofing, applied hot. The three layers of cloth were then laid in the usual manner, each layer being mopped before the application of the succeeding layer. This waterproofing was carried up the sides of the parapet wall to the top of the ties and directly across all expansion joints, so that the waterproofing was in reality continuous from one end of the bridge to the other. At the expansion joints one additional layer of the saturated fabric was laid across and folded in the expansion joint beneath a copper flashing, similarly laid, over which the three layers of waterproofing were placed. A fold was provided in the waterproofing at the joints to provide for expansion and the entire joint filled with the Minwax hard waterproofing, poured hot. A detail of this expansion joint is illustrated in the accompanying sketch.

As a protection to the waterproofing the Johns-Manville asphaltic mastic, mixed with torpedo washed gravel, was applied hot in two ¾-in. layers over the entire area of the waterproofing. In order to avoid injury to the waterproofing by the hot mastic, one ply of Johns-Manville asbestos felt was first laid over the entire area of the waterproofing. An opening was left in the mastic directly over the center of each expansion joint. This opening was filled with the Minwax hard waterproofing compound. The Johns-Manville asphaltic mastic was used for protection in preference to brick or concrete, since our experience elsewhere with this mastic, under ballast, indicates that it does not crack and in reality forms a secondary waterproofing surface on which the drainage readily passes to the downspouts. G. J. RAY,

Chief Engineer, D., L. & W. R.R.

Hoboken, N. J., Feb. 24, 1916.

Lime To Correct Acid Sludge in Imhoff Sewage Tanks

Sir—In Doctor Imhoff's article entitled "Eight Years of Imhoff Tank Design and Operation—II," published in *Engineering News*, Jan. 13, 1916, very decided views are expressed by the author on the inadvisability of adding lime to the sludge compartment of an Emscher tank to correct acidity as an aid in obtaining good sludge. As our results in Philadelphia are quite contrary to these ideas, I think it would be of interest to your readers to know our experience, which is as follows:

The Pennypack Creek sewage-works at Philadelphia treat sewage collected from about 4,500 persons, two-thirds of whom are housed in the County Prison, House of Correction and the Home for Indigent. In these institutions breakfast usually consists of bread and coffee; supper of bread and tea, or cocoa; dinner of bread and soup, or meat stew, with occasionally roast meats, vegetables, dried fruits, etc. With this diet it is self-evident that the suspended matter of the sewage would be highly carbonaceous and would contain more than normal amounts of fat in appreciable-sized pieces.

The sludge deposited in the Imhoff tank upon decomposition apparently produces more carbonic and fatty acids than the ammonia produced by the hydrolysis of the nitrogenous bodies can neutralize.

During the early operation of the plant the gas bubbling up through the vent was inodorous and could be ignited, and the sludge withdrawn contained considerable quantities of gas, showing that decomposition was going on in the sludge compartment. But the sludge withdrawn up to last spring was gray in color, sticky and heterogeneous in structure, not free from odor and dried slowly on the sand beds.

An examination of the sludge from one of the tanks showed it to contain 2,200 p.p.m. acid (by phenolphthalein as indicator). Barrels were set up over the vent, with a spigot near the bottom, and lime water added therefrom to the sludge by a pipe extending from above the water level to mid-depth of the sludge.

The lime could not be added at a very uniform rate, owing to clogging of the spigot, but about 10 bu. of lime a month were added to the two tanks, which together receive 1,250,000 gal. of sewage a day.

Each successive withdrawal of sludge thereafter showed less acid content and an improvement in quality, and in the late fall considerable quantities of sludge were removed from each tank to avoid applying sludge to the drying beds during freezing temperatures. This sludge was all of good quality, practically neutral in reaction, black, granular, free flowing, filled with gas, inodorous and quick drying.

As the tanks were put in service in 1912 and no really good sludge obtained until after the application of lime, it is only reasonable to presume that lime was the cause. However, in order to demonstrate whether the addition of the lime tended to weaken the good bacteria, as Doctor Imhoff has suggested, no lime has been added all winter, and it will be of interest to find what character of sludge will be obtained in the spring.

W. L. Stevenson,
Assistant Engineer, Sewage Disposal.

City Hall, Philadelphia, Feb. 11, 1916.

Ductile Steel for Safety

Sir—The editorial note appearing in *Engineering News*, Jan. 20, 1916, under the caption "Ductile Steel for Safety" calls attention to a subject that cannot be brought too forcibly to the attention of all engineers—particularly of those in responsible charge of specifications for steel and for metal reinforcement.

There is no reason why the steel used to reinforce the tension side of a reinforced-concrete bridge floor-beam or stringer should not be of metal as reliable as the metal in the lower chord of a short-span bridge. The writer knows of an instance where a whole carload of steel supplied for use as metal reinforcement for a concrete-bridge floor was rejected because its elongation was 10% or less. (The specifications required an elongation of 25%.) The plea was made in an effort to have the material accepted, that the use of similar material was "almost universal" and that, if the material was "accepted with a warning," no more similar material would be supplied. The rejection was insisted upon, no further discussion resulted, and no more steel was supplied, for use in concrete-bridge floors, that did not have an elongation of 25%.

Similar action in like circumstances would soon put an end to the "almost universal use" of bars "short" in elongation and therefore lacking in reliability. Contractors are willing to supply any grade; in fact, they prefer the soft steel, because of the greater ease with which it can be handled in the field.

About the best single indicator of steel suitable for bridge members subjected to a wide variation in stress intensity or to shock is its elongation. Steel having an elongation of about 25% and possessing the usually attendant properties is suitable for the lower chord of a short span bridge, the tension side of a slab or beam of a reinforced-concrete bridge floor or for any bridge member subjected to shock or to a wide range of stress.

To one familiar with the treatment of metal reinforcement in the field during the various operations preliminary to the pouring of the concrete, the use of steel of mild grade seems almost imperative. Hard steel is difficult to handle and to bend to prescribed shape; it requires experts to prevent serious injury. The ease with which mild steel can be bent to desired shape is really an index of its capacity to do useful work; that is, mild-steel bars having an elongation of 25% are sometimes deformed with the intention of increasing their efficiency as metal reinforcement. After deformation these bars are found to have an elongation of only 10% and frequently much less. The process of deformation has reduced the potential elongation of the material from 25% to 10%. This loss in potential elongation is an index of the capacity of the metal to give service.

W. H. Yates.

52 Wall St., New York City, Feb. 4, 1916.

Sinking-Fund Delusions

Sir—Permit me to make another effort to indicate clearly my position in regard to one feature of the much misunderstood subject of so-called "depreciation" (cost of periodic renewals of plant)—this in answer to your editorial comments on my letter in your issue of Jan. 27.

In the case cited in my letter, assuming an average life of plant of 40 years, if we require the consumers to pay on the "straight-line" basis, we charge to operating cost each year for 40 years $2.50 for each $100 worth of plant periodically to be renewed. If we require the consumers to pay only on the basis of a 4% sinking fund, we charge to operating cost each year for 40 years only $1.05 for each $100 worth of plant periodically to be renewed. This saving in cost of operating, which presumably will be reflected in the service rate under commission regulation, certainly is not a "trick in words or bookkeeping." It is a very real difference of $1.45 per $100 on all the plant to be periodically renewed.

If the annual charge is made on the "straight-line" basis and this $2.50 per $100 is carried into a sinking fund to accumulate at 4% compound interest, there should be in the fund at the end of 40 years $238.09, or $138.09 in excess of the requisite $100.

If the full $2.50 per $100 per year is put into plant extensions in lieu of isolated investment in a sinking fund, then the earnings on $1.05 of the $2.50 so invested yearly added to the 40 payments of $1.05 will show a total of $100, as required, provided the rate of earnings on these extensions does not fall below an average of 4%. The $1.45 also invested in plant extensions under the same conditions will have created a surplus in form of plant equal to $138.09 for each $100 worth of plant to be renewed, as estimated at the beginning of the 40 years. This surplus in the form of additions to plant will have been paid for from the rates. Here I am not involving

the question as to return on investment, but I am considering the one item of operating cost—namely, maintenance of plant.

It is never to be forgotten that estimates of renewal reserves based upon expected life, involving physical decay or wear, obsolescence and inadequacy, are necessarily liable to error and should therefore be so liberal as to afford full protection to the investment. Furthermore, such estimates should year by year be checked in the light of accumulated experience.

If it is understood that the "straight-line" method is advocated because of this uncertainty as to final cost and is also intended to cover other contingencies and if the assumed "average life" is subject to amendment as indicated, then we have a proposition which is defensible. Let it then be seen that justification of the "straight-line" method on the ground of prudent provision against contingencies is quite different from attempting to justify this larger charge on mathematical grounds or by arguing that the sinking-fund scheme is a "trick in words or bookkeeping."

As to the courts, it is true that their decisions cannot change facts and are supposed only to interpret the laws for the guidance of all concerned. We cannot certainly claim that these interpretations are always correct, for we are constantly obliged to face widely divergent court interpretations on the same law. Even the Knoxville decision, to which you refer, is not in turn given a uniform interpretation by lawyers and courts. As to the inability of the courts to "alter physical facts," it is to be mentioned that some of their opinions and decisions do rest upon an alteration of physical facts. For instance, in one of the most important and far-reaching public-utility decisions of the United States Supreme Court the learned judge who wrote the opinion gave as the chief reason for declaring unconstitutional one feature of the law in question the "fact" that an internal pressure equal to that exerted by a column of water 2½ in. high would burst a cast-iron gas main. So while I am prepared to accord to the courts all proper respect, I also hold that we engineers can instruct the courts as to much that is involved in their "opinions."

This view has *recently* received support at the hands of Judge Prouty, formerly chairman of the Interstate Commerce Commission and now chairman of the Valuation Board, and by Mr. Daniels, a member of the Interstate Commerce Commission.

ALEX. C. HUMPHREYS,
President, Stevens Institute of Technology.

Hoboken, N. J., Jan. 31, 1916.

[Under the conditions assumed by President Humphreys the actual loss of capital through "depreciation"—which we agree it is better to call periodic renewals or retirements—averages $2.50 for each year of the 40. To keep the accounts aligned with the physical facts, President Humphreys should charge to operating cost each year as nearly as possible the actual annual decrement of capital—the annual increment of "depreciation," some still call it. If the straight-line computations are followed, this will be $2.50 in the hypothetical case given. He should also diminish the stated value of the property by the amount so repaid—through changing the ledger value of investment or in some equivalent way. If the operating cost is charged with only $1.05 each year, the difference, say $1.45, must still be recovered from the business in some way. Practically, this is done by holding the stated earning value of the property, or the ledger value, etc., at the original figure—in spite of the capital actually sacrificed. Then the customers pay $1.05 as an acknowledged repayment against depreciation and $1.45 as pseudo-interest, which is really an acknowledged contribution toward depreciation. This is the "trick in words and bookkeeping" that we have referred to. Obviously, if the annual contributions are computed on a direct-repayment basis (like the straight-line scheme), it is not pertinent to discuss their accumulation in a sinking fund or to treat their reinvestment as different from fresh capital.

The attempt of many persons to separate the study of so-called "depreciation" contributions from the consideration of a rate-basis worth in most discussions of rate making, as shown in President Humphreys' letter, constitutes one of the stumbling blocks in the path toward better mutual understanding of all parties concerned. The two matters are so intimately connected, as we have partly indicated, that the full and proper procedure in one direction cannot be scrutinized until it is seen what has been done in the other.—Editor.]

Masonry Reinforced with Metal Nothing New

Sir—The article on the Bone patent, in *Engineering News* of Feb. 10, 1916, has been read with a great deal of interest. The germ of practically all use of reinforcing metal in masonry structures is found in the experimental arch constructed by Sir Isambart Brunel, described in "Civil Engineering," by Henry Law, p. 132, Part II, published in Weale's series, 1869. The advertisement to this book is dated July, 1869. The copy I have has been in my possession since March, 1874.

Under the direction of the late Alfred L. Rives, M. Am. Soc. C. E., the writer made use of the method known as "hoop-iron bond" for strengthening the tension side of brick lintels and slabs, notably in constructing brick-masonry foundation for track scales in the marsh at Mobile, about 1872. The experiment made by Brunel was in 1837-38.

The writer was familiar with the articles in *La Construction Moderne* in 1894 and wondered how it was possible for the Patent Office to grant a patent to Bone at the time it was granted. Another paper, by I. K. Brunel, in the paper of the Institution of Civil Engineers, 1838, p. 20, on "Use of Hoop-Iron Bonding in Brick and Cement Beams," is of interest.

J. G. WALKER.

U. S. Engineer Office, Nashville, Tenn., Feb. 12, 1916.

Arch Centers Lagged with Sods

Sir—Herewith are two photographs of a unique method of constructing arch centers, as practiced by Ecuadorian masons in the interior of the country. The arch in question carries an irrigating ditch across a cañon.

The span of the arch is 25 ft.; rise, 12 ft.; thickness at crown, 2 ft.; width, 3½ ft.

As the photograph shows, a triangular framework is first made, using 6-in. diameter timbers of eucalyptus, fastened together by ropes made from split cactus, or maguey, leaves. Then the lagging is formed of sods cut

A 25-FT. ARCH IN ECUADOR; SOD-LAGGED CENTERS STRUCK AUTOMATICALLY BY SHRINKAGE

from the fields. The sods, called *chamba,* consist of the roots of wild grass and earth. They are 6 in. thick by 2 ft. long and 1 ft. wide, are packed on the triangular framework and shaped to the curve of the arch ring. After the arch is built, the sods are left until they dry out and shrink, when they are easily removed.

The photograph was taken two weeks after the arch was built and just before the centering was removed.

H. F. Howe,
Chief Engineer, Guayaquil & Quito Ry. Co.
Guayaquil, Ecuador, Dec. 10, 1915.

Water-Gas Tar for Treating Wood-Paving Blocks

Sir—As interest in the subject of specifications for creosoted wood blocks is at present so great, the article appearing on page 204 in the Feb. 3 issue of *Engineering News* will no doubt be read very generally. It is therefore regrettable that Dr. Von Schrenk's admirable paper should not have been printed in full.

To the casual reader it would appear from the opening paragraph that radical changes in the specifications for creosoted wood-block pavements had received indorsement at the meeting of the American Society of Municipal Improvements held in Dayton, Ohio, Oct. 12 to 14, 1915, but such was not the case. While it is true that the subcommittee did adopt a specification that contained some radical changes from those formerly existing in that society, these radical changes were not accepted by the general committee. The general committee's report to the society recommended that action be deferred until the 1916 meeting, so that all interested might have sufficient time to give their views upon this important subject and thus enable the subcommittee to draft a specification more acceptable to all parties concerned.

We are not able to judge from the data given in the short résumé of Dr. Von Schrenk's paper whether the use of refined water-gas tar as a preservative material is condemned or not. We are inclined to the former conclusion, however, as the only definite reference to the examination of wood blocks treated with that material indicates a failure. Quoting the words of the article, "The failure of blocks in a pavement in Charleston, S. C., was traced to the use of a straight water-gas oil and very poorly penetrated blocks." This appears to the writer as an unjust condemnation, when it is acknowledged by Dr. Von Schrenk that "poor penetration, particularly of the sapwood," is a primary cause of failure.

It is regrettable that such an extensive and painstaking investigation of this subject as Dr. Von Schrenk undoubtedly made did not include a greater yardage of blocks treated with water-gas tar, as millions of such blocks have been laid in the past nine years, to the writer's knowledge, and not a single instance of failure attributable to the preservative used has been reported.

An article by Henry Welles Durham, formerly Chief Engineer, Bureau of Highways, Borough of Manhattan, New York City, appeared in *Engineering News,* Dec. 30, 1915. It gave some very definite observations of a recent examination of test pavements laid on Second Ave., New York City, with creosoted wood blocks that were treated with varying quantities of different types of preservative oils, including two sections treated with water-gas tar—one with 16-lb. treatment, the other with 20-lb. treatment.

Two significant statements are contained in this report: "No difference in appearance is noticeable between the differently treated sections except in extremely hot weather, when there is a slight evidence of oil on the surface of the blocks treated with light oil, but bleeding has practically stopped"; and "Nothing in the present appearance of any of these stretches of pavement indicates that under ordinary conditions any variation of treatment of the blocks had been employed."

Quoting again from the résumé of Dr. Von Schrenk's paper, we find the statement, "The criterion to be used in specifying the preservative ought to be only availability and cost." Under such a criterion water-gas tar would certainly be placed first; and owing to its long and favorable performance, serious consideration should be accorded water-gas tar and a competent specification for it included in all general specifications for wood-block preservative oils.

J. S. Miller, Jr.,
Assistant Chemist, Barber Asphalt Paving Co.
Maurer, N. J., Feb. 9, 1916.

Safe-Stairway Specifications

The number of casualties that occur to persons slipping on stairways is surprising to anyone who has not studied the statistics. In a paper by H. W. Mowery, just published by the American Museum of Safety, New York City, it is stated that there were last year in the single Borough of Manhattan 170 deaths from falls on stairs and sidewalks. During the same period the total deaths from fires were 65, from surface cars 53 and from subway and elevated cars 24. The total of 142 deaths from these four agencies, commonly supposed to involve such danger to human life, was thus considerably less than the number of deaths due to falling on stairs and sidewalks.

The lesson Mr. Mowery draws from these statistics is that it is well worth while for the engineer responsible for the design and construction of stairways to specify nonslipping treads, to properly proportion the treads and risers and to use materials that will not produce a glazed and slippery surface. One can see on the sidewalks of every city and in stores, public buildings, etc., a multitude of cases where these simple precautions have been neglected.

Many of the common errors of construction that occur even when an attempt is made to secure safety are mentioned in Mr. Mowery's paper. A specially noteworthy one is the common practice of covering stair treads with grooves filled with some anti-slip composition running parallel to the treads. In descending such stairs it is difficult in certain lights to distinguish the edge of the step from the grooves parallel with it. It is just as easy to make the grooves for the anti-slip composition run diagonally with the top of the step and thereby save many dangerous falls.

At the close of his paper Mr. Mowery presents the report of a committee of engineers of the Pennsylvania R.R. appointed by S. C. Long, General Manager, to recommend specifications for safe stairways. This report is as follows:

For stairways used by the traveling public the sum of two risers and one tread shall be between 25 and 26 in. No tread shall exceed 13 in. or be less than 12 in., and no riser shall be more than 6¾ in. nor less than 6 in. No corrugated nor ribbed tread over ⅛ in. in depth shall be used on any stair. Noncorrugated, anti-slip treads of lead and aluminum or carborundum, iron and alundum or carborundum, or concrete and alundum or carborundum are recommended for any width of not less than 8 in. on tread, including nosing. Nosings having a projection not exceeding 1 in. are recommended for use on treads. All measurements are to be from top to top of treads and face to face of risers. All stairways to have handrails at each side. Top of handrail to be 3 ft. above top of tread, measured vertically at nosing. All stairways requiring more than 16 risers should have a landing 4 ft. wide, and in general, the single flights should be kept down to 10 or 12 risers where possible. The height of risers and width of treads for stairways used by the traveling public shall be the same for all such stairways throughout in any one building.

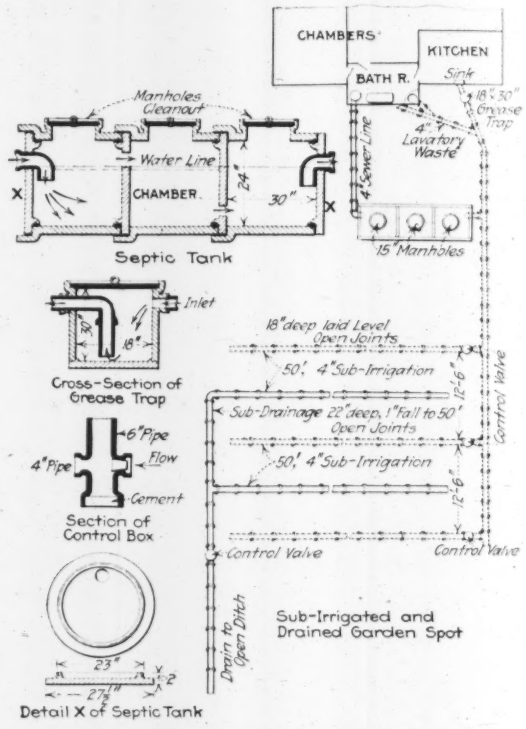
For stairways in office buildings and in places used jointly by employees and the general public the sum of two risers and one tread shall be between 24 and 25 in. No riser to exceed 8 in. or less than 7 in. No tread to exceed 11 in. or be less than 10 in. All stairways to have handrails at sides. Top of handrail to be 3 ft. above top of tread measured vertically at nosing. Noncorrugated, anti-slip treads of lead and alundum or carborundum, iron and alundum or carborundum, or concrete and alundum or carborundum are recommended for a width of not less than 8 in. on tread, including nosing. All stairways requiring more than 18 risers should have a landing 4 ft. wide, and in general, the single flights should be kept down to 10 or 12 risers where possible. The height of risers and width of treads for stairways in office buildings and in places used jointly by employees and the general public shall be the same for all such stairways throughout in any one building.

For stairways in signal towers, shops and places used by employees, but not for the use of the public, the above rules for stairways in office buildings and places used jointly by the employees and the general public shall obtain, with the exception that the width of tread shall be not less than 9 in. or more than 11 in.

Single-House Sewage Disposal Plant, Lewisville, Ark.

A vitrified-clay septic tank and sub-irrigation tiles beneath a drained garden are used for the treatment of the sewage from the residence of J. B. Burton, of Lewisville, Ark. The layout and details of the plant are shown by the accompanying illustration.

The septic tank is in three sections, or chambers, each with an estimated daily capacity of 50 gal. The kitchen and bathroom wastes do not pass through the septic tank,



SEPTIC TANK AND SUB-IRRIGATION SYSTEM FOR HOUSE AT LEWISVILLE, ARK.

but go directly to the sub-irrigation system. There is a grease trap on the kitchen-sink outlet. The branch tile distributors and also the underdrain are provided with "control valves."

The septic tank shown was patented Apr. 8, 1913. It is called the adjustable-capacity septic tank (abbreviated to "Adj-Cap" as a trade name), because the number of sections, or chambers, may be increased to correspond with the volume of sewage to be treated. Thus in a plant installed for the Texas State Tuberculosis Colony No. 1, at Carlsbad, Tex., a 60-section tank of this type is in use.

The system of sewage treatment here described was installed under the supervision of the Texarkana Pipe Works, of Texarkana, Ark.

News of the Engineering World

New York Military-Engineering Courses Grow

The second and third lectures in the course on elementary military engineering arranged by the New York engineering societies were held on Feb. 21 and 28. To accommodate all who desire to attend, one series has been arranged for Mondays at 5:30 p.m. and two for 8 p.m.; one of the evening lectures is held in the house of the American Society of Civil Engineers and the other meetings are in the Engineering Societies' Building.

The lectures on Feb. 21 were broken into by a number of speakers seeking recruits for the Plattsburg training camps held by the United States Army. The great interest shown by the New York engineers has led to plans for weekly drills. Two of the city armories will be opened and instruction furnished in March, April and May, under officers of the state militia, on squad and company work, close and extended order, guard duty, target practice, use of arms, packing, etc. It is aimed to give such a preliminary course as will allow more time at the Plattsburg army camp for practice with engineer troops.

Maj. Gen. Leonard Wood, commanding the Department of the East, U. S. A., in an address at the third lecture outlined the system desired by the army in the development of reserve troops. He cited the experience in the wars of the Revolution, 1812 and 1863 as showing that the volunteer system has never been a success. The state militia was described as a weak arm in spite of good men and officers, because of poor equipment, small units, lack of Federal control and the necessity of converting from state to Federal system in event of war. He advocated complete federalization of the state militias and general military service for boys, on the Swiss or Australian plan.

The lecture on Feb. 21, by Capt. R. T. Coiner, Corps of Engineers, U. S. A., was on reconnaissance and the allied topics of surveying and mapping. Great dependence has come to be placed by commanders on maps, since battle fronts have extended far beyond the observation of one man. Many features not seen on ordinary maps are required—character, size and grades of roads and dimensions of bridges; railways, telephone and telegraph lines; canals, rivers and lakes, with data on currents, banks and fords; complete topography, with all obstacles like fences, walls and hedges; street plans of towns, grazing areas, water-supply facilities, etc.

In America all military maps are made to uniform scales of 1, 3, 6 or 12 in. to the mile. Main courses are laid out by magnetic bearings and backsights; distances are taken by pacing, time of horse travel, or revolutions of a wheel; side shots are estimated. In action, maps are made by and behind the advance cavalry screen of a large army. Road maps come first, giving main-column routes and parallel roads. Work of individual sketchers is tied together later by rough triangulation. If there is time, tracings are made; if no time is available, the field papers are inked and prints taken direct. Blueprints are preferred, with brown next in favor. Brownprints with white background are the most useful; bromide papers are used for night printing. Each engineer battalion carries a reconnoissance surveying outfit, a lithographing press and a hektograph.

The third of the regular lectures was by Capt. T. M. Robins, Corps of Engineers, U. S. A., and dealt largely with field fortifications. The requirements of cover were given as 3 ft. of earth for light fire, 12 ft. against field artillery and 30 ft. or more for heavy artillery. The construction of rifle trenches, saps, communicating and supporting trenches of all degrees of completeness was described and their function explained, largely as laid down in the Corps of Engineers' Field Manual and other publications of the army. It was shown how most of this work is done by the line troops, and the special functions of the engineers were outlined. For the simpler intrenchments the line troops now all carry special small shovels and picks, but the engineer battalions are well equipped with real tools—shovels, pick mattocks, bars, saws, sandbags, spikes, etc. A large amount of accessory trench equipment has to be provided by the engineers—drains, sumps, water supply, guide signs, lamps, and all the various obstacles in front. The work of engineers in demolition, largely involving the use of explosives, was explained; the growing use of trinitrotoluol as a substitute for dynamite, etc., was mentioned.

More Water for New York

A 250,000,000-gal. per day addition to the Catskill water-supply of like amount, already developed by the Board of Water-Supply of New York City, has been approved by the Board of Estimate, and the plans for the project will soon go to the State Conservation Commission for its approval. A court action to stop the project was started, but was almost immediately dismissed. The estimated cost of the Schoharie addition is $22,000,000, and the time required for the construction has been estimated as 8 years.

The Ashokan reservoir, formed by the Olive Bridge dam across Esopus Creek, impounds water from a drainage area of 257 sq.mi. The Schoharie drainage area, north of the Esopus gathering ground, covers 314 sq.mi. Still other drainage areas are included in an ultimate water-supply scheme involving a total watershed of 914 sq.mi., which it is estimated will yield 760,000,000 gal. a day through a series of extraordinarily dry years, if sufficient storage is provided. The Catskill acqueduct, leading from the Ashokan reservoir to New York City, has a daily capacity of 500,000,000 gal.

To make the Schoharie supply available, a dam will be built at Gilboa and a tunnel 16⅔ mi. long driven from the reservoir thus formed, to discharge into the Ashokan reservoir.

J. Waldo Smith is engineer, and Alfred D. Flinn is deputy chief engineer of the Board of Water-Supply.

Argentine Business Conditions

Engineering News receives frequent inquiries as to the possibilities of engineering work in South America, from engineers who are desirous of seeking opportunities there. The following extract from a letter written by a well-known American engineer, who is at present connected with the Buenos Aires office of a United States export company, is instructive:

It may interest you to learn something of the present construction situation in this country. The Government, whose revenues are mainly derived from duties on imports, which are of course greatly curtailed on account of the war, is having a great deal of difficulty in meeting its ordinary expenses, and public works are practically at a standstill. The large port works being constructed by the well-known English firm C. H. Walker & Co. have been stopped completely. Payment for this work was being made in bonds; and as the market for these has become extremely limited, Walker was unable to carry on the work. The situation in regard to the irrigation works in Rio Negro, under construction by the Southern Ry., and those under construction by the Central Argentine Ry. is similar, and they have been, or will be, stopped for the same reasons. There are rumors that American interests may take up these works, but as yet nothing has come out of this.

A few small irrigation projects built by the Government itself are still underway, but water-works and so on are quite dead. The large post-office building remains in the condition in which it was one year ago. The steelwork, amounting to 6,000 tons, has been erected and since then repainted several times, but nothing else has been done. This building, located near the port, may be seen from miles out in the channel leading to the city and is a mute evidence to every traveler of the embarrassed position in which the war has placed this country. Until ocean freight rates become more normal and material can arrive in greater volume and until better prices may be obtained for the products of the country, business will be dull.

Bridge and building work is at a standstill. The railroads are doing almost nothing, and many good-sized buildings that were in construction in Buenos Aires at the beginning of the war have remained in the state in which they were left at that time. Exposed steelwork is rusting away, and brick walls are in danger of falling, on account of mortar washing out. About a month ago a 15-story reinforced-concrete building, the lower portion of which runs through from one street to another, forming a handsome arcade, was opened to the public with many ceremonies, but this work was in hand for a long time before the war commenced. A few other buildings, such as the Stock Exchange, are approaching completion. A contract has been signed to construct about 60 mi. of narrow-gage railroad connecting the northernmost town in Argentina—La Quiaca—with Tupiza in Bolivia, but I doubt whether it will be pushed very actively.

Wisconsin Engineers Meet

The eighth annual meeting of the Engineering Society of Wisconsin was marked by active and prolonged discussion of the relations of engineers and contractors and the respective merits of closed and open specifications for paving. The meeting was held in the Engineering Building of the University of Wisconsin, at Madison, Wis., on Feb. 25 and 26.

J. C. Pinney, Superintendent of Bridges and Buildings at Milwaukee, urged that the engineer should give the contractor full information, through the plans and specifications, as to the work required. Otherwise, the contractor cannot intelligently estimate the cost. If "clean" sand is specified, the contractor may figure on using a local sand containing 5 or 6% of loam and find later that the engineer will not allow more than 2 or 3%.

That the engineer should not act as arbiter between client and contractor was the position taken by Prof. D. W. Mead, Madison, and F. G. Simmons, Director of Public Works at Milwaukee. The latter stated that, with full intent to be impartial, the engineer will naturally and involuntarily give greater weight to the interests of his client. In reply to a suggestion that a contractor should not be required to guarantee work for more than one year, he remarked that the purpose of the guarantee is to provide against defective materials and workmanship and that one year is not always sufficient to develop such conditions.

One great difficulty, according to Mr. Simmons, is the tendency to severe competition between contractors. Although prices for materials are higher than a year ago, many recent bids are 6 to 7% lower. For one large job the contractor admitted that he allowed only 5% for profit, which is unreasonably low; and under such conditions the engineer must expect trouble in getting work done to his satisfaction. Mr. Mead also urged that specifications should be fair to both parties and not put all responsibility on the contractor and all authority upon the engineer.

Open Specifications for Paving

A paper on the satisfactory experience with open specifications for asphalt paving, by H. W. Adams, Mayor of Beloit, Wis., precipitated a brisk and protracted discussion. When the city planned large contracts for paving in 1915, strong representations were made that a certain material was so superior in quality that it could not compete in price with others and that the city could well afford to use this material in spite of its higher price. Instead of this, open specifications were employed, requiring the materials to meet tests as to quality. The lowest bids proved to be for the material for which a high price had been claimed to be necessary. Mr. Adams considered the use of open specifications as good public policy, while it tends to avoid excessively high bids and any tendency toward graft.

That open specifications are not advisable or economical for small towns which have no municipal chemist or laboratory was the view taken by P. H. Connolly, city engineer of Racine; that they are undesirable for large cities, especially in admitting various grades of asphalt into competition, was the attitude of F. G. Simmons, Director of Public Works of Milwaukee, in a lengthy discussion strongly favoring a particular grade of asphalt.

The discussion then turned on the law requiring bids to be asked for different kinds of paving on any job, the property-owners then having an opportunity to select which they prefer. A few speakers favored this as conducing to competition, but the weight of opinion was strongly opposed to such a haphazard method of selection, which may tend to cheapness, but not to economy or efficiency. The paving should be selected by authority competent to determine what is best suited to the conditions.

Other Subjects and Proceedings

Bituminous paving was dealt with in two papers. One was on asphaltic concrete pavement, by W. F. Reichardt, consulting engineer, Watertown, Wis. In the discussion Prof. L. S. Smith exhibited samples, including one from a bituminous concrete laid in Washington in 1873. The other paper was on the surface treatment of old macadam paving, by T. R. Valk, Chicago. He advocated cleaning the old surface, applying a bituminous coating and covering this with a layer of stone chips. Care must be taken to clean the surface thoroughly.

A paper on "State Inspection of Public Utilities" was read by C. M. Larsen, Chief Engineer of the Wisconsin Railroad Commission, and the Wisconsin building code was discussed by S. J. Williams, State Inspector. At the annual dinner Prof. D. W. Mead gave an illustrated address on the Dayton flood and the proposed flood-protection works.

Iowa Engineers Talk Ethics

Coöperation, legislation, commercialism and ethics in relation to engineers and engineering were the predominating subjects at the twenty-eighth annual meeting of the Iowa Engineering Society, held at Des Moines, Feb. 16 to 18. This line of discussion was opened with an address by Prof. F. H. Newell, University of Illinois, on the needs and advantages of coöperation in the advancement of the profession to a position where it receives proper public appreciation and recognition. Dean W. G. Raymond, of the State University of Iowa, described the organization of the new Association of Engineers, whose aims are avowedly commercial. Some of its features did not meet with his approval, and he questioned whether the real interests of the profession would be advanced by such a movement.

On the other hand, the report of a Committee on Professional Ethics and Standard Fees, presented by K. C. Gaynor, Sioux City, Iowa, expressed the view that the organization of the association mentioned indicated appreciation of the fact that engineers as a class are wage earners and must organize to obtain suitable pay and recognition. It was shown in the discussion that many engineers on important work in Iowa are paid by the day at rates about the same as those paid for skilled labor. At the annual dinner the principal address was on "Engineering Ethics," by Hon. George Cosson, State Attorney General.

The Committee on Legislation approved a bill for the licensing of engineers, but opposed an architects' licensing bill drawn on the lines that have proved so objectionable in Illinois. A move is now being made to secure the coöperation of the Iowa chapter of the American Institute of Architects in preparing a bill for the licensing of both professions. The chairman spoke of the difficulty of getting engineers to attend the legislative meetings. After setting forth to the legislative committee that he represented the opinions of a large body of engineers, it was discouraging to be able to show only three engineers present to support his position.

Sanitary matters received a good share of attention in the more strictly technical subjects. In a paper on "Infiltration of Ground Water into Sewers," C. P. Chase, Clinton, Iowa, stated that it was almost impracticable to make sewers absolutely tight. It is difficult to guarantee a leakage of less than 10,000 gal. per mi. in ordinary soil or 20,000 gal. in water-bearing sand. Where water can get in, of course sewage can leak out. Mr. Chase also reported for a committee appointed to confer with the State Board of Health in regard to the form of plans and specifications submitted for water-works, sewers and sewage-disposal plants. The forms recommended have been approved by the board; but as they have not been formally adopted, the plans actually submitted for approval "come in every conceivable shape and drawn on everything from butchers' paper to bristol board."

The care and operation of sewage-disposal plants in Iowa was discussed by Dean Marston, Iowa State College, who stated that many such plants are so badly neglected as to be wholly or partly out of service. Many plants are built too small, owing to inadequate resources, but he suggested that engineers can help to remedy this by refusing to design filters for excessive rates of flow. The State Board of Health should refuse to approve plans calling for such rates. He considered that a city council should place entire charge of the plant in the hands of a competent superintendent, preferably the city engineer.

Dean Marston stated also that the Cameron septic-tank patents are again being made the basis of demands for royalties in Iowa, in spite of the fact that the suit against Knoxville, Iowa, was decided in favor of the city by the United States Supreme Court. For this reason a new Septic Process Protective League is being organized by the League of Iowa Municipalities, to obtain the legal aid of the attorney general.

In regard to road and street matters a report on roads and pavements was presented by C. H. Young, of Muscatine; the relation of subgrade construction to cracks in concrete paving was discussed by A. H. Gilliland, Indianola, and a paper on bituminous pavement was read by M. L. Patzig, Des Moines. An interesting paper on "A Comparision of Iowa Paving Law Assessments," by C. A. Roby, Waterloo, showed the variations in assessments under identical conditions in different cities. One session also was set aside for a joint meeting with local good-roads associations.

Among other papers were "Railway Signaling," by G. H. Kilpatrick, Waterloo, and "The Old and New Turbines of the Hydro-Electric Plant of the State University," by R. E. Hutchins, Iowa City. The boiler code of the American Society of Mechanical Engineers was reviewed by S. M. Woodward, Iowa City, and the society adopted a resolution advocating its general use. Prof. A. H. Ford, State University of Iowa, described the nitrogen-filled incandescent lamp for street lighting. In speaking of the advantage of this lamp C. P. Chase, Clinton, stated it had taken the manufacturers six months to devise a bug-proof lamp suitable for cities along the Mississippi River, where vast quantities of bugs and flying insects constitute a serious problem in the street-lighting scheme.

The society elected the following: President, M. G. Hall, consulting engineer, Centerville, Iowa; vice-president, C. H. Young, City Engineer, Muscatine, Iowa; secretary, J. H. Dunlap, Iowa City, Iowa.

A Sewage-Disposal System for Los Angeles County, California, has been proposed as a result of agitation against the joint sewage-disposal project for Pasadena, South Pasadena, and Alhambra. The Board of Supervisors of Los Angeles County voted on Jan. 12 to appoint a commission of five engineers to report on the subject.

Electrification of Railway Terminals at St. Paul and Minneapolis has been proposed by H. M. Byllesby, of the Minneapolis General Electric Co., who has presented plans for the work to President Louis W. Hill, of the Great Northern Ry., and his assistant, Ralph Budd. The project is under consideration in connection with the construction of the new union station at St. Paul.

A Strike of Anthracite Miners may take place on Apr. 1 when the present contract between the anthracite operators and the miners expires. The miners have presented to the operators a series of demands which include an increase of 20% on all wage rates paid in the anthracite coal field, an 8-hr. day for all day labor, time and a half for overtime and double time for Sundays and holidays. Further demands are

recognition of the United Mine Workers and an agreement that all coal shall be weighed and paid for on a run-of-mine basis with no deductions for dirt and rock in the coal.

Extra Pay for Lock-Gate Work—Washington reports say that the claim for $2,394,480 or more of extra pay by the McClintic-Marshall Construction Co., Pittsburgh, is likely to be one-third successful. The claim was referred by vote of Congress to Governor Goethals for report. It is now said that Governor Goethals has reported his recommendation that the company be paid about $714,000. The claim was for additional work required in producing the extra-high grade of workmanship demanded by the inspectors, which the company said was far ahead of that called for by the specifications or by the requirements of the case. The original claim was reviewed in "Engineering News," Apr. 9, 1914, p. 806.

An Inland Waterway along the New Jersey coast extends from Cold Spring Inlet, near Cape May, to the northerly end of Barnegat Bay, near Bay Head, about 114 mi. The waterway has a minimum depth of 6 ft., a bottom width of 100 ft. and is 98% completed. It is under control of the Board of Commerce and Navigation of New Jersey, of which B. F. Cresson, Jr., is Chief Engineer. In the construction of this waterway more than 3,000,000 cu.yd. of material has been excavated, and the outlay for dredging has been $318,251. For the extension of this waterway northward a contract has been let for the first section of a canal to extend from Bay Head to the Manasquan River.

A Conference on drainage and related subjects will be held at the University of Illinois under the auspices of the department of civil engineering, Mar. 8 to 11, 1916. Land reclamation, flood protection, levee construction and the importance of such work to the agricultural interests of the state are subjects which will be discussed by engineers and contractors who have specialized in such work. The practicability will be considered of having an annual short course in drainage engineering under the direction of Prof. F. H. Newell, head of the department of civil engineering and formerly director of the United States Reclamation Service. All who are interested in the subjects mentioned are cordially invited to attend. The department of civil engineering, Urbana, will send full information on request.

Cast-Iron and Steel Water Pipe were in competition at St. Louis, Mo., on Feb. 25, when the Water Department received bids for supplying 27,000 ft. of 36-in. pipe. The East Jersey Pipe Co., New York City, bid $193,680 for lock-bar riveted steel pipe, coated with mineral rubber and wrapped with burlap. The United States Cast Iron and Foundry Co., Philadelphia, bid $207,648, or about $28.80 per ton, on cast-iron pipe. In addition, there is about $17,000 worth of special castings to be added to the bid on cast-iron, which was practically all included in the bid on steel. Bids were also received on laying each class of pipe, and a canvass of all the propositions indicates that cast-iron in place will cost about $35,600 more than the steel. The president of the Board of Public Service has not yet decided which material will be adopted.

A Banquet in the Mill Creek Sewer was held in St. Louis on Feb. 19, in honor of Mayor Kiel and to celebrate the practical completion of the Mill Creek tunnel. The hosts were the contractors—the Carter Construction Co., and Norton, Head & Dineen. The banquet was one of the most elaborate and spectacular ever given in connection with construction work and it is estimated by the local press to have cost $5,000, of which more than $2,000 was spent in transforming this section of the tunnel into a banquet hall. About 200 ft. of the tunnel adjacent to one of the shafts was bulkheaded off for the occasion; a special wood floor was laid and covered with tar paper and tarpaulin. A large ventilating fan was installed in the bulkhead, and toilets and wash basins were provided. The concrete work was painted white, and a large number of electric lights was installed, while steam was piped into the shaft for steam radiators, and gas for cooking ranges.

Ontario Municipal Tramways—The annual convention of the Ontario Hydro-Electric Railway Association was held at Toronto on Feb. 16, practically all the important municipalities in the province being represented. Resolutions were adopted calling upon the government to restrict the export of power to the United States, to prevent the existing power companies from increasing their capital for the purpose of making extensions to their plants, urging the dominion and provincial governments to grant subsidies to the municipalities to aid in the construction of the projected radial lines and asking the Ontario Government to pass legislation authorizing the Hydro-Electric Commission to proceed with surveys and acquire rights-of-way for the hydro-radial railways. Both governments were asked not to grant new franchises or powers for railway construction to any private interests. Sir Adam Beck, chairman of the Hy[illegible] Electric Commission, reviewed the work of that body an[illegible]tended that the proposed hydro-radial lines would be a pa[illegible] proposition from the outset. He considered it essenti[illegible]hat the Chippewa development should proceed simultane[illegible]y with the construction of the radial lines. The follow[illegible] officers were elected: Honorary president, Sir Adam Beck[illegible] honorary vice-presidents, Hon. I. B. Lucas and W. K. McNa[illegible]s; president, J. W. Lyon (Guelph); vice-presidents, May[illegible] Thomas Church (Toronto), A. F. Wilson (Markham), Ma[illegible] H. R. Stevenson (London), Mayor J. J. Duffus (Peterbo[illegible] Mayor W. F. Walters (Hamilton), J. B. Ryan (Guelph) an[illegible] H. Clay (Windsor); secretary, T. J. Hannigan (Guelph); [illegible]easurer, G. P. Hamilton (Guelph).

To Make Soil-Pressure Tests—Experiments on the distribution of pressures through soils are to be conducted at the Government farm near Washington by the Office of Public Roads and Rural Engineering, United States Department of Agriculture. A specially designed apparatus will be installed which will permit earth-pressure measurements being taken in a large number of positions during a single application of the load. This feature is regarded as an important one, and it is hoped that an unnatural rearrangement of the soil from previous load applications in various positions will be thus avoided. Earth-fills up to 10 ft. thick may be accommodated by this apparatus, and it will have lateral dimensions sufficiently large to permit obtaining the full distribution of the load in all directions. Very comprehensve plans have been made for a practical solution of this problem, and it is hoped that definite results will be forthcoming within a short time. The uncertainty of the distribution of pressures through soils, both vertically and horizontally, has long been recognized by the engineering profession, and several isolated experiments have been made with laboratory apparatus to gain knowledge on this subject. In magnitude the problem is such that its practical solution can be attempted only by the most thoroughly equipped experiment stations. In application the final solution will be important to all branches of engineering, and it has seemed desirable that the department should devote a part of its experimental resources to a most thorough and conclusive research of this nature.—Circular of Department of Agriculture.

Sewerage Improvenemts at Nashville, Tenn., are recommended in a report submitted to the City Commission by James Cameron, an accountant of New York City, and H. M. Brinckerhoff, acting for Barclay, Parsons & Klapp, consulting engineers, New York City. The report deals with the Health Department at Nashville and is a part of the general survey of the affairs of that city being conducted by the men named. The report states that the present sewerage system has a large preponderance of trunk sewers and lacks corresponding development of laterals. This appears to be due to the fact that the proceeds of bond issues have been used for trunk sewers and that the expectation of building laterals with money from current funds has not been realized. The report suggests that surveys and estimates be made to determine the cost of continued extensions to the sewerage system with particular attention, first of all, to those districts showing the greatest need. It is even suggested that the expenditure of $90,000 in a single district which now has a high typhoid death rate and a large amount of pellagra would remove the district from a position of danger to one of safety. The report also suggests city supervision of privies, the regular collection of night-soil and the granting of authority to the Health Department to close contaminated wells, together with coöperation by the water-works department in the way of making extensions which will facilitate the closing of wells. The geological formation at Nashville is such as to increase the danger of contamination of wells from privies and cesspools. A shallow layer of soil rests upon limestone containing many crevices. The seepage from privies and cesspools passes through seams and crevices in the limestone to springs and wells. The latter, it appears, have been extensively used by a public which has assumed that because water is clear and otherwise agreeable to the physical senses it is therefore free from all danger of contamination.

PERSONALS

Mr. F. R. Williams has been appointed Assistant City Engineer of Cleveland, Ohio, in charge of new paving. He succeeds Joseph Bayne.

Mr. A. J. Cleary, Assoc. M. Am. Soc. C. E., has been appointed Second Assistant Engineer to City Engineer O'Shaughnessy, of San Francisco.

Mr. H. W. Shimer has been appointed First Assistant Engineer to City Engineer O'Shaughnessy, of San Francisco, to fill the vacancy caused by the death of Loren E. Hunt.

Mr. R. W. Gelder, Assoc. M. Am. Soc. C. E., has been appointed City Engineer of Greeley, Colo., to fill the vacancy created by the resignation of O. F. Shattuck, M. Am. Soc. C. E., now located in Indianapolis, Ind.

Mr. F. D. Holbrook, M. Am. Soc. C. E., has left the Government service to become associated with R. A. Miller under the firm name of Miller & Holbrook, Consulting Engineers, with offices in the Millikin Building, Decatur, Ill.

Mr. Joseph Bayne, Assistant City Engineer of Cleveland, Ohio, who has been in the employ of the city for more than 30 years, has resigned to take charge of the paving work of the Cleveland Railway Co. He will take up his new duties Mar. 1.

Mr. Harry K. Miller, who for the past two years has been Assistant Engineer in charge of a new survey of the streets and boundaries of the town of Montclair, N. J., and the monumenting connected with this work, has resigned from that position.

Mr. W. L. Hemmings has been appointed Smoke Commissioner of the City of Cleveland, Ohio, by Public Safety Director A. B. Sprosty. Mr. Hemmings will receive an annual salary of $1,800. Under the previous administration the position paid a salary of $2,200.

Mr. H. F. Wardwell, formerly Home Office Engineer of the Detroit Steel Products Co., Detroit, Mich., has been appointed General Sales Manager of the company, succeeding Mr. P. A. Smith, recently resigned. Previous to his present connection Mr. Wardwell was in the employ of the United States Gypsum Co.

Dr. F. W. Cunningham, of Orange, N. J., has severed his connection with the research and development work of the Newark, N. J., plant of the General Electric Co., to join the engineering-research staff of the Powdered Coal Engineering and Equipment Co., of Chicago, Ill. He will be located in Chicago after Mar. 1.

Mr. B. L. G. Rees, formerly Assistant Engineer with the New York Public Service Commission on the new subway work, will sail on Mar. 4 for Antofagasta, Chile. He will accompany Mr. Huntington Adams, M. Am. Inst. M. E., who is making the trip to investigate an engineering project in that vicinity. Mr. Rees will have charge of the survey work connected with the investigation.

Mr. T. V. Stephens, for the past two years City Engineer of Cadillac, Mich., has been appointed City Manager of Cadillac. He succeeds Mr. O. E. Carr, now City Manager of Niagara Falls, N. Y. Mr. Stephens graduated at the Pennsylvania State College in civil and sanitary engineering in 1912. Previous to his residence in Cadillac he was connected with the Division of Sewerage Investigation, Designs and Plans of the City of Cincinnati.

Mr. Samuel A. Greeley, Assoc. M. Am. Soc. C. E., until recently Assistant Engineer with the Sanitary District of Chicago, Ill., on investigations, designs and construction of sewers and sewage-treatment plants and formerly in charge of the construction and operation of the refuse incinerator of Milwaukee, Wis., has opened an office at 64 West Randolph St., Chicago, for the practice of hydraulic and sanitary engineering. Mr. Greeley graduated at Harvard University in 1903 and later took a bachelor of science degree in sanitary engineering at the Massachusetts Institute of Technology, after which for several years he was employed as assistant engineer with Rudolph Hering and George W. Fuller. He has spent considerable time in Europe investigating and reporting upon the practice of sanitary engineering there and has made a report on sanitary conditions of the City of Caracas, Venezuela.

OBITUARY

James Driscoll, a well-known contractor, died at his home in Brookline, Mass., on Feb. 20.

George R. Ray, President of the Manistee Iron Works Co., of Manistee, Mich., died on Jan. 3. Mr. Ray was prominently identified in the development of equipment, particularly the vacuum pan, for the manufacture of salt.

Col. J. M. Wilkinson, of Valdosta, Ga., died in Augusta, Ga., on Feb. 16. He was born in Liberty County, Ga., and was about 63 years old. He built the Valdosta Southern Ry. from that city to Madison, Fla., which in 1908 became part of the Georgia & Florida system. Colonel Wilkinson was made vice-president of the latter road; and when, in 1913, receivers for it were appointed, he was made one of them.

T. Ryan, a prominent contractor of Seattle, Wash., died on Feb. 10. He had been a resident of that city for 33 years. During the early part of that period he erected some of the largest buildings in Seattle. In later years he turned his attention to street paving, his best-known work being the repaving of Second Ave., Seattle. He served on the King County Commission and the City Council for several years.

George H. McPhetres, member of the Lynn Municipal Council, in charge of streets and highways, died at his home in that city on Feb. 25. Mr. McPhetres was born in Harmony, Me., and moved to Lynn in 1886, where he entered the employ of the street department, with which he was connected in all for 27 years. He was commissioner of streets and highways when Lynn changed its charter in 1912, under which he was elected a member of the Municipal Council. At the time of his death he was serving his second term in that capacity.

Bryce R. Blair, who as chief engineer constructed the Jefferson branch of the Erie R.R. and was also engaged in the construction of the Bloomsburg division of the Delaware, Lackawanna & Western R.R., the Broad Top Mountain R.R. at Taxton, Penn., and a large section of the Albany & Susquehanna division of the Delaware & Hudson R.R., died at Carbondale, Penn., on Feb. 11. Mr. Blair was 83 years of age and a native of Glasgow, Scotland. He served for seven years as City Engineer of Carbondale and was largely influential in the development of the city's municipal improvements.

Peter James Cunneen, Manager of the Lassig plant of the American Bridge Co., at Chicago, died on Feb. 13. He was born in Adrigoe, Ireland, in 1859. In 1878 he entered the employ of the old American Bridge Co. and remained with the company and its successors until the formation of the present company. Mr. Cunneen was Secretary of the Chicago Forge and Bolt Co. and later of the American Bridge Works. After the formation of the present American Bridge Co. he served as purchasing agent in charge of the Chicago district and was later made Manager of the Lassig plant, which position he held for about 15 years, up to the time of his death.

ENGINEERING SOCIETIES

COMING MEETINGS

AMERICAN ROAD BUILDERS' ASSOCIATION.
Feb. 28 to Mar. 3. Annual convention in Pittsburgh. Secy., E. L. Powers, 150 Nasau St., New York City.

CENTRAL RAILROAD CLUB.
Mar. 9. Annual meeting in Buffalo. Secy., H. D. Vought, 95 Liberty St., Buffalo.

VERMONT SOCIETY OF ENGINEERS.
Mar. 9. Annual meeting in Burlington. Secy., George A. Reed, Montpelier.

NEW ENGLAND RAILROAD CLUB.
Mar. 14. Annual meeting in Boston. Secy., W. E. Cade, 683 Atlantic Ave., Boston.

WISCONSIN ELECTRICAL ASSOCIATION.
Mar. 16-17. Annual convention in Milwaukee. Secy., George Allison, Milwaukee.

AMERICAN RAILWAY ENGINEERING ASSOCIATION.
Mar. 21-23. Annual meeting in Chicago. Secy., E. H. Fritch, 900 So. Michigan Ave., Chicago.

NATIONAL RAILWAY APPLIANCES ASSOCIATION.
Mar. 21-23. Annual meeting in Chicago. Secy., B. V. Crandall, 1824 Lytton Building, Chicago.

ILLINOIS GAS ASSOCIATION.
Mar. 22-23. Annual meeting in Chicago. Secy., H. H. Clark, 136 So. Harvey Ave., Oak Park, Ill.

Maine Society of Civil Engineers held its annual meeting at Augusta in February and elected the following officers for 1916: President, Paul D. Cargent; vice-president, Harry E. Green; secretary, Frank E. Pressey; directors, I. W. Barbour, John Calvin Stevens, E. C. Jordan, T. W. Clark, S. A. Moulton.

Canadian Mining Institute is holding its annual meeting (Mar. 1 to 3) in Ottawa. The president-elect is Arthur A. Cole, mining engineer for the Temiskaming & Northern Ontario Railway Commission, of Cobalt. The secretary is H. Mortimer-Lamb, Room 911, Ritz-Carlton Hotel, Sherbrooke St., West, Montreal.

Western Society of Engineers—A joint meeting of the Chicago sections of the American Institute of Electrical Engineers and the Illuminating Engineers Society and of the

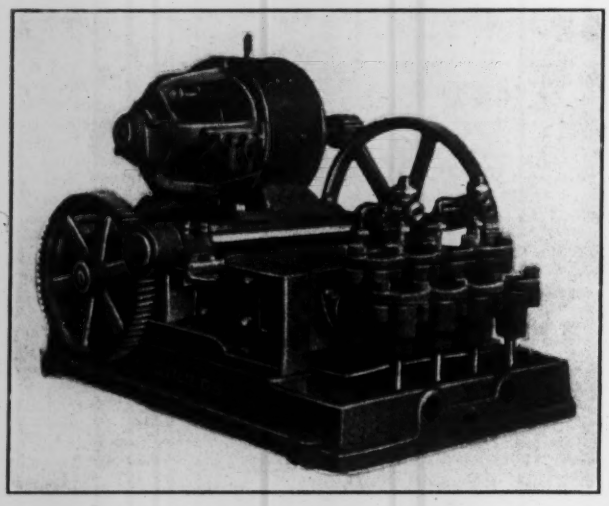
electrical section of the Western Society of Engineers was held in the rooms of the latter in Chicago on Feb. 28. The general subject of the evening was street illumination, three papers being presented, by J. R. Cravath, A. C. King and F. A. Vaughn.

Joint Meeting of Electricals and Electrochemicals, New York sections, will be held in the auditorium of the Engineering Societies Building on Friday, Mar. 10, at 8:15 p.m. The subject of the meeting will be "Electrolysis," and two papers will be presented. On behalf of the American Institute of Electrical Engineers a paper will be presented by Burton McCollum, of the Bureau of Standards, and G. H. Ahlborn, entitled "The Influence of Frequency of Alternating or Infrequently Reversed Current on Electrolytic Corrosion." On behalf of the American Electrochemical Society a paper will be presented by Prof. William H. Walker, of the Massachussetts Institute of Technology, entitled "Electrolytic Corrosion of Metals."

Appliances and Materials

High-Pressure Oil Pump

A motor-driven pump, shown in the accompanying illustration has been designed by the Goulds Manufacturing Co., Seneca Falls, N. Y., for forcing oil under the bearings of machinery. The pump is a single-acting horizontal quadruplex with ⅜x4-in. cylinders; it has a capacity of 2 gal. per



QUADRUPLEX OIL-PRESSURE PUMP

min. against 1,200 lb. per sq.in. In the base is a 10-gal. reservoir. The cylinders are brass lined; plungers are of hardened tool steel and crossheads of bronze. Connecting-rods are of cast steel, babbitted on the crank end and bronze bushed at the crosshead.

* * *

Combination of Pump Designs

The American Steam Pump Co., Battle Creek, Mich., maker of the "Marsh" and "American" types of boiler-feed and other pumps, has now put out the "American-Marsh" design, which combines the general features of the "Marsh" line with the "American" valve. The main valve is a balanced-piston type with expansion rings; the auxiliary valve is of a semirotary disk type, mechanically actuated by a lever arm connected to the piston crosshead. The new line covers heating-system, boiler-feed and vacuum pumps.

* * *

Improved Cores for Cast-Iron Pipe

The commercial development of the type of cores, for use in casting iron pipe, invented by the late H. M. Fletcher, of Cincinnati, has been taken up by the concern, Fletcher Cores, 820 Gwinne Building, Cincinnati. The first steps in this invention were taken with the coöperation of the late Matthew Addy, who operated pipe foundries at Newport, Ky., and Addyston, Ohio. It was designed to obviate the necessity of a thick combustible layer and the inequalities of wall found with hay-rope cores.

A vented cast-iron core bar, larger than the ordinary, is used. It is spirally wound with strips of embossed paper and in some cases with a follower of hemp twine. A coating of loam is then put on. There is claimed to be a saving of 40 to 60% in the amount of loam used and a small saving in cost of paper strips over hay rope. The time required to form and dry the core and to clean the cast pipe is reduced.

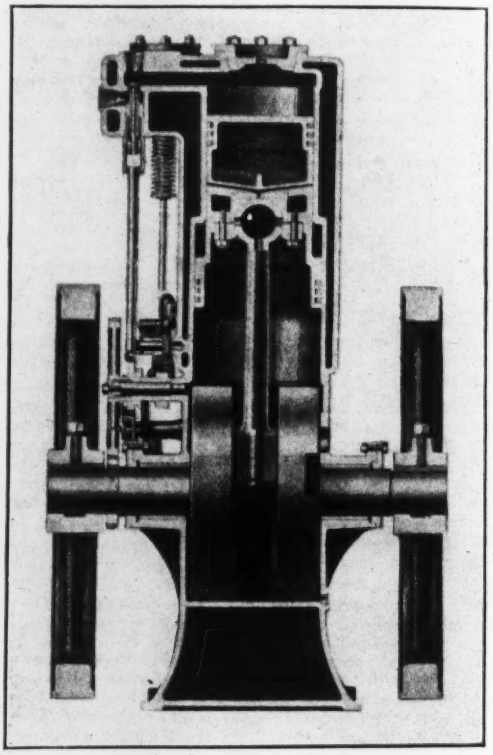
Fletcher cores were first put into practical operation in the plant of the Warren Foundry and Machine Co., Phillipsburg, N. J. and have recently been adopted by the National Cast Iron Pipe Co., Birmingham, Ala. The largest pipe yet cast with these cores is 16 in. in diameter.

* * *

Vertical Gasoline-Driven Air Compressor

A new simple design of portable self-contained gasoline-driven air compressor has been completed by the Lyon-Atlas Co., Indianapolis, Ind. It is built for pressures of up to 200 lb. per sq.in., and in three sizes—for 50-, 100- and 150-cu.ft. piston displacement. The arrangement, shown in the accompanying section, is very simple and compact.

The engine is of single-cylinder four-cycle type. By use of a differential piston and cylinder only one piston rod is required. The air is admitted through a port in the case and a registering port in one crank disk; it is first compressed to about 20 lb. in the crank case and flows into a receiving passage and into the annular space of the second cylinder. To unload the compressor, the air-admission valve between the crank case and second cylinder is held open at a predetermined pressure. Most of the working parts are in easy reach and plain view on the outside. By disconnecting the



VERTICAL DOUBLE-STAGE GASOLINE-DRIVEN AIR COMPRESSOR

outside pipe and removing the casing bolts the top of the casing may be swung back on hinges, exposing crank, crank disks, piston and casing interior.

The engine is hit-or-miss governed. While the standard engine is intended for gasoline, it can be adapted to kerosene or any gas. To secure cold water around the compressor cylinder and warm water in the engine jackets, the cooling-water flow is from compressor to engine. The three sizes develop 9, 18 and 25 hp.; the shipping weights are 1,500, 2,900 and 4,000 lb.

* * *

Insulating and Rust-Preventing Paint

Under the trade name "Rostnicht" the Billings-Chapin Co., Cleveland, Ohio, is making an insulating and anti-corrosion paint for steel. This paint consists of zinc chromate and inert pigments (among which portland cement is commonly supposed to figure largely) carried in a varnish vehicle. It is claimed to be impervious to water, brine, oil, fumes and gases. It is furnished in seven colors.